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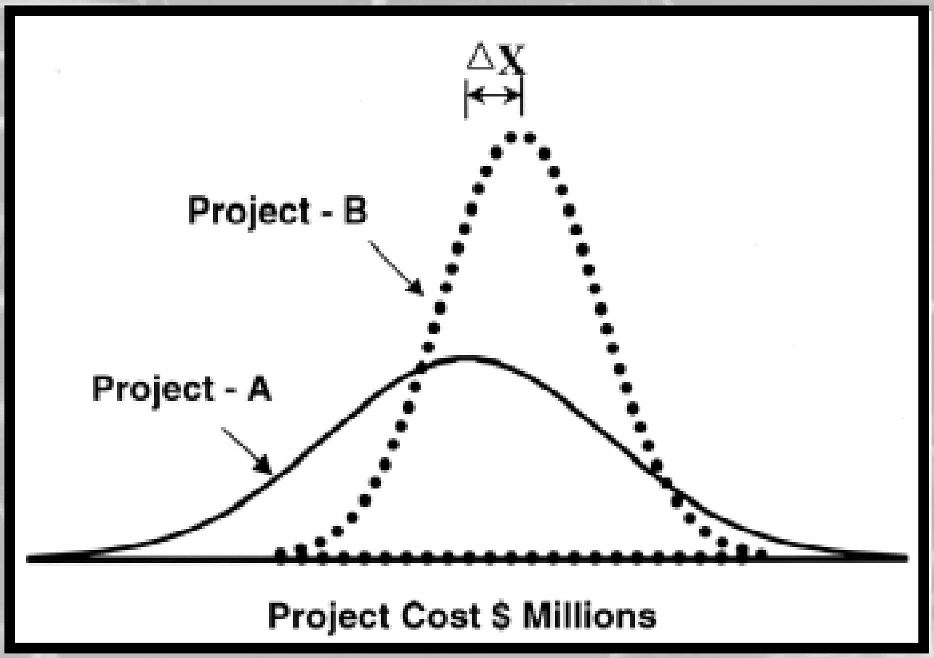
**Federal Highway
Administration**

Life-Cycle Cost Analysis in Pavement Design

- In Search of Better Investment Decisions -



**Pavement Division Interim Technical Bulletin
September 1998**

This *Interim Technical Bulletin* presents technical guidance and recommendations on good/best practices in conducting Life-Cycle Cost Analysis (LCCA) in pavement design. The *Bulletin* will be of interest to State highway agency personnel responsible for conducting and/or reviewing pavement design LCCAs.

To reinforce this publication, the FHWA Office of Engineering, Pavement Division, in cooperation with the Office of Technology Applications, offers LCCA technical support through FHWA Demonstration Project No.115, *Probabilistic LCCA in Pavement Design* (DP-115). DP-115 is a free 2-day workshop that demonstrates good/best practices in performing life-cycle cost analyses for pavement design. This workshop is available, upon request, to State highway agencies.

This publication and DP-115 support the FHWA's response to the Transportation Equity Act for the 21st Century legislative mandate to develop recommended procedures for conducting LCCA on National Highway System projects.

Henry H. Rentz, Director
Office of Engineering
Federal Highway Administration

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16. Abstract

This *Interim Technical Bulletin* recommends procedures for conducting Life-Cycle Cost Analysis (LCCA) of pavements, provides detailed procedures to determine work zone user costs, and introduces a probabilistic approach to account for the uncertainty associated with LCCA inputs. The *Bulletin* begins with a discussion of the broad fundamental principles involved in an LCCA. It discusses input parameters and presents simple examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendation on acceptable ranges for the value of time as well as discount rates. It explores the use of sensitivity analysis in traditional LCCA approaches. User costs are a combination of delay, vehicle operating costs, and crash costs. Each of these cost components is explored and procedures are presented to determine their value. Given the power and sophistication of today’s computers and software, simulation techniques such as Monte Carlo are recommended for incorporating variability associated with LCCA inputs into final results.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	ac
ha	hectares	2.47	acres	mi^2
km^2	square kilometers	0.386	square miles	
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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ABBREVIATIONS

ADT	Average Daily Traffic
AADT	Average Annual Daily Traffic
BAMS	Bid Analysis Management System
B/C	Benefit/Cost Ratio
CFR	Code of Federal Regulations
CPI	Consumer Price Index
DP-115	Demonstration Project 115
DOT	Department of Transportation
EUAC	Equivalent Uniform Annual Cost
FARS	Fatal Accident Reporting Systems
FHWA	Federal Highway Administration
GAO	General Accounting Office
HCM	Highway Capacity Manual
HERS	Highway Economic Requirements System
HMAC	Hot-Mix Asphalt Concrete
HOT	High Occupancy Toll
HOV	High Occupancy Vehicle
IRI	International Roughness Index
IRR	Internal Rate of Return
ISTEA	Intermodal Surface Transportation Efficiency Act
LCC	Life-Cycle Cost
LCCA	Life-Cycle Cost Analysis
LTPP	Long-Term Pavement Performance
mi/h	miles per hour
MPO	Metropolitan Planning Organization
NCHRP	National Cooperative Highway Research Program
NHS	National Highway System
NPV	Net Present Value
NPW	Net Present Worth
OIG	Office of Inspector General
OMB	Office of Management and Budget
OST	Office of the Secretary of Transportation
PennDOT	Pennsylvania Department of Transportation
PCCP	Portland Cement Concrete Pavement
pcplph	passenger cars per lane per hour
PV	Present Value
PSR	Present Serviceability Rating
RSL	Remaining Service Life
SHA	State Highway Agency
SLC	Single-Lane Closure
SOV	Single Occupancy Vehicles
TEA-21	Transportation Equity Act for the 21st Century
TLTWO	Two-Lane Two-Way Operation
VMT	Vehicle Mile(s) of Travel
vph	Vehicles Per Hour
V/C	Volume-to-Capacity Ratio
VOC	Vehicle Operating Cost
WZ	Work Zone

EXECUTIVE SUMMARY

This *Interim Technical Bulletin* provides technical guidance and recommendations on *good practice* in conducting Life-Cycle Cost Analysis (LCCA) in pavement design. It also introduces Risk Analysis, a probabilistic approach to describe and account for the uncertainty inherent in the process. It deals specifically with the technical aspects of the long-term economic efficiency implications of alternative pavement designs. The *Bulletin* is directed at State highway agency (SHA) personnel with responsibility for conducting and/or reviewing pavement design LCCAs.

Purpose of LCCA

LCCA is an analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing alternative investment options. It does not address equity issues. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value (the lowest long-term cost that satisfies the performance objective being sought) for investment expenditures.

LCCA Requirements

The National Highway System (NHS) Designation Act of 1995 specifically required States to conduct life-cycle cost analysis on NHS projects costing $25 million or more. Implementing guidance was provided in Federal Highway Administration (FHWA) Executive Director Anthony Kane's April 19, 1996, Memorandum to FHWA Regional administrators.
The implementing guidance did not recommend specific LCCA procedures, but rather it specified the use of good practice.

The FHWA position on LCCA is further defined in its Final Policy Statement on LCCA published in the September 18, 1996, *Federal Register*. FHWA Policy on LCCA is that it is a decision support tool, and the results of LCCA are not decisions in and of themselves. The logical analytical evaluation framework that life-cycle cost analyses fosters is as important as the LCCA results themselves. As a result, although LCCA was only officially mandated in a very limited number of situations, FHWA has always encouraged the use of LCCA in analyzing all major investment decisions where such analyses are likely to increase the efficiency and effectiveness of investment decisions whether or not they meet specific LCCA-mandated requirements.

The 1998 Transportation Equity Act for the 21st Century (TEA-21) has sinced removed the requirement for SHA's to conduct LCCA on high-cost NHS useable project segments. However, the congressional interest in LCCA is continued in the new requirement that the Secretary of Transportation develop recommended LCCA procedures for NHS projects.

Bulletin Format

The *Interim Technical Bulletin* discusses the broad fundamental principles involved in LCCA and it presents widely accepted procedures used in setting up and conducting LCC analysis.

It also discusses input parameters, the variability and inherent uncertainty associated with them, and provides recommendations on acceptable ranges for a variety of parameters. It presents examples of traditional LCCA in a pavement design setting. It then provides a detailed, rational highway capacity-based approach for determining work zone user delay, vehicle operating, and crash costs associated with alternative pavement design strategies. It explores the use of sensitivity analysis in traditional LCCA approaches and introduces a probabilistic-based risk analysis approach to account for the variability of inputs. Several microcomputer software programs are available for conducting deterministic LCCA on routine pavement rehabilitation projects. There are also powerful microcomputer-based risk analysis software programs currently on the market that work well in conjunction with standard computer spreadsheet applications. The appendix to this *Interim Bulletin* includes a discussion of supporting computer software and additional LCCA resource documents.

LCCA Procedures

Life Cycle Cost (LCC) analysis should be conducted as early in the project development cycle as possible. For pavement design, the appropriate time for conducting the LCCA is during the project design stage. The LCCA level of detail should be consistent with the level of investment. Typical LCCA models based on primary pavement management strategies can be used to reduce unnecessarily repetitive analyses.

LCCA need only consider differential cost among alternatives. Costs common to all alternatives cancel out, are generally so noted in the text, and are not included in LCCA calculations. Inclusion of all potential LCCA factors in every analysis is counterproductive; however, all LCCA factors and assumptions should be addressed, even if only limited to an explanation of the rationale for not including eliminated factors in detail. Sunk costs, which are irrelevant to the decision at hand, should not be included.

LCCA Principles of Good Practice

The LCCA analysis period, or the time horizon over which alternatives are evaluated, should be sufficient to reflect long-term cost differences associated with reasonable design strategies. While FHWA's LCCA Policy Statement recommends an analysis period of *at least* 35 years for all pavement projects, including new or total reconstruction projects as well as rehabilitation, restoration, and resurfacing projects, an analysis period range of 30 to 40 years is not unreasonable.

Net Present Value (NPV) is the economic efficiency indicator of choice. The Uniform Equivalent Annual Cost (UEAC) indicator is also acceptable, but should be derived from NPV. Computation of Benefit/Cost (B/C) ratios are generally not recommended because of the difficulty in sorting out cost and benefits for use in the B/C ratios.

Future cost and benefit streams should be estimated in constant dollars and discounted to the present using a real discount rate. Although nominal dollars can be used with nominal discount rates, use of real/constant dollars and real discount rates eliminates the need to estimate and include an inflation premium. In any given LCCA, real/constant or nominal dollars must *not* be mixed (i.e., all costs must be in real dollars or all costs must be in nominal dollars). Further, the discount rate selected must be consistent with the dollar type used (i.e., use real cost and real discount rates or nominal cost and nominal discount rates).

The discount rates employed in LCCA should reflect historical trends over long periods of time. Although long-term trends for real discount rates hover around 4 percent, 3 to 5 percent is an acceptable range and is consistent with values historically reported in Appendix A of OMB Circular A-94.

Performance periods for individual pavement designs and rehabilitation strategies have a significant impact on analysis results. Longer performance periods for individual pavement designs require fewer rehabilitation projects and associated agency and work zones user costs.

While most analyses include traditional agency costs, some do not fully account for the SHA engineering and construction management overhead, especially on future rehabilitations. This can be a serious oversight on short-lived rehabilitations as SHAs design processes lengthen in an era of downsizing.

Routine, reactive type annual maintenance costs have only a marginal effect on NPV. They are hard to obtain, generally very small in comparison to initial construction and rehabilitation costs, and differentials between competing pavement strategies are usually very small, particularly when discounted over 30- to 40-year analysis periods.

Salvage value should be based on the remaining life of an alternative at the end of the analysis period as a prorated share of the last rehabilitation cost.

User Costs

User costs are the delay, vehicle operating, and crash costs incurred by the users of a facility and should be included in the LCCA. Vehicle delay and crash costs are unlikely to vary among alternative pavement designs between periods of construction, maintenance, and rehabilitation operations. Although vehicle operating costs are likely to vary during periods of normal operations for different pavement design strategies, there is little research on quantifying such Vehicle Operating Cost (VOC) differentials under the pavement condition levels prevailing in the U.S.A. The *Technical Bulletin* therefore focuses strictly on work zone user cost differences between alternatives.

User costs are heavily influenced by current and future roadway operating characteristics. They are directly related to the current and future traffic demand, facility capacity, and the timing, duration, and frequency of work zone-induced capacity restrictions, as well as any circuitous mileage caused by detours. Directional hourly traffic demand forecasts for the analysis year in question are essential for determining work zone user costs.

As long as work zone capacity exceeds vehicle demand on the facility, user costs are normally manageable and represent more of an inconvenience than a serious cost to the traveling public. When vehicle demand on the facility exceeds work zone capacity, the facility operates under forced-flow conditions and user costs can be immense. Queuing costs can account for more than 95 percent of work zone user costs with the lion’s share of the cost being the delay time of crawling through long, slow-moving queues.

Different vehicle classes have different operating characteristics and associated operating costs, and as a result, user costs should be analyzed for at least three broad vehicle classes: Passenger Vehicles, Single-Unit Trucks, and Combination Trucks.

User delay cost rates are probably the most contentious of all user cost inputs. While there are several different sources for the dollar value of time delay, the recommended mean values and ranges for the value of time (Aug 96 $) shown in the table below appear reasonable. It is important to note that commercial vehicles support higher values of travel time delay rates and that passenger vehicles, particularly pickup trucks, represent both commercial and noncommercial use.

Recommended values of time.

Vehicle Class	**$ Value Per Vehicle Hour**	
	Value	**Range**
Passenger Vehicles	$11.58	$10 to 13
Single-Unit Trucks	18.54	17 to 20
Combination Trucks	22.31	21 to 24

Work zone crash cost differentials between alternatives are very difficult to determine because of the lack of hard statistically significant data on work zone crash rates and the difficulty in determining vehicle work zone exposure. However, default dollar value ranges associated with fatal and nonfatal injury highway crashes are included.

Risk Analysis

LCCA, as a minimum, should include a sensitivity analysis to address the variability within major analyses input assumptions and estimates. Traditionally, sensitivity analysis has evaluated different discount rates or assigned value of time, normally evaluating a *best* and *worst* case scenario. The ultimate extension of sensitivity analysis is a probabilistic approach, which allows all significant inputs to vary simultaneously.

The *Interim Technical Bulletin* advocates the use of a probabilistic approach to LCCA that incorporates analysis of the variation within the input assumptions, projections, and estimates. The prevailing term used in private industry for a probabilistic approach is *Risk Analysis*. Risk analysis is a technique that exposes areas of uncertainty, typically hidden in the traditional *deterministic* approach to LCCA, and it allows the decision maker to weigh the probability of the outcome actually occurring. The risk analysis approach combines probability descriptions of uncertain variables and a computer simulation technique, generally know as Monte Carlo Simulation, to characterize uncertainty. Monte Carlo simulations randomly draw samples from the individual inputs consistent with their defined distributions to calculate thousands, even tens of thousands, of *what if* outcomes. With enough samples, the program can define an overall composite NPV probability distribution for each alternative — one that shows the entire range of possible outcomes and the likelihood that any particular outcome will actually occur. Given the power and sophistication of today's computers and software, the FHWA strongly endorses the use of techniques, such as Monte Carlo simulation, for incorporating variability associated with LCCA inputs into final results.

CHAPTER 1. INTRODUCTION

The purpose of this *Interim Technical Bulletin* is to provide technical guidance and recommend *good practice* in conducting Life-Cycle Cost Analysis (LCCA) in pavement design and to introduce Risk Analysis, a probabilistic approach, which describes the uncertainty inherent in the process. The primary audience for this *Bulletin* is State highway agency (SHA) personnel responsible for conducting and/or reviewing LCCA of highway pavements. This includes State pavement design engineers and pavement management engineers, as well as district or area supervisors responsible for selecting pavement type and rehabilitation strategies.

SCOPE

The *Interim Technical Bulletin* recommends specific procedures for conducting LCCA in pavement design and discusses the relative importance of LCCA factors on analysis results. In the interest of technical purity, the discussion includes all relative LCCA factors, even though not all elements influence the final LCCA results to the same degree. The *Bulletin* first addresses the broad fundamental principles involved in LCCA; this is followed by presentation of the widely accepted procedures used to set up and conduct LCC analysis. It discusses input parameters and presents examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters and recommends acceptable ranges for a variety of parameters. It explores the use of sensitivity and introduces a risk analysis approach to account for the variability of inputs. Finally, there is a discussion of supporting computer software. The appendix lists additional LCCA resource documents specific to pavement design.

While the issue of equity is a highly significant consideration in any public investment decision, it is not part of the economic efficiency issue. This *Interim Technical Bulletin* deals specifically with the technical aspects of the long-term economic efficiency implications of alternative pavement designs.

LCCA results are a useful decision support tool, but they are not decisions in and of themselves. Frequently, the analytical evaluation that such analysis fosters is as important as the LCCA results. As a result, SHAs are encouraged to conduct LCCA in support of all major investment decisions.

APPLICATION

Fundamental principles of economic analysis have broad application. In general, this *Interim Technical Bulletin* presents generic concepts that may be applied to areas other than pavements. For example, LCCA may be applied to establish funding levels, allocate resources among program areas, and prioritize project selection.

LEVEL OF DETAIL

The relative influence of individual Life-Cycle Cost (LCC) factors on analysis results may vary from major to minor to insignificant. The analyst should ensure that the level of detail incorporated in an LCCA is consistent with the level of investment decision under consideration. There comes a point of diminishing returns as more and more cost factors are incorporated in an LCCA. For example, slight differences in future costs have a marginal effect on discounted present value. Including such factors as this unnecessarily complicates the analysis without providing tangible improvement in analysis results. Including all factors in every analysis is frequently not productive. The difficulty in capturing some costs makes omitting them the more prudent choice — particularly when the effect on the LCCA results is marginal at best.

In conducting an LCCA, analysts should evaluate all factors for inclusion and explain the rationale for eliminating factors. Such explanations make analysis results more supportable when they are scrutinized by critics who are not pleased with the analysis outcome. This *Interim Technical Bulletin* does not provide guidance on determining the appropriate extent of LCCA on specific projects.

LCCA DRIVING FORCES

The current FHWA position on pavement-related LCCA has its roots in the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, which specifically required consideration of “the use of life-cycle costs in the design and engineering of bridges, tunnels, or pavement” in both Metropolitan and Statewide Transportation Planning. Additional direction came in January 1994 with Executive Order No.12893, “Principles for Federal Infrastructure Investments,” which requires systematic analysis of benefits and costs when making infrastructure investment decisions. It also requires that the costs be measured and discounted over the full life cycle of each project. Further, an Office of Inspector General/Government Accounting Office (OIG/GAO) 1994 *Highway Infrastructure* report on cost comparison of asphalt versus concrete pavements reviewed in the Federal Highway Administration (FHWA) Region 4 States made specific recommendations on the FHWA’s need to provide additional technical guidance on LCCA.(1)

In addition, the National Highway System (NHS) Designation Act of 1995 specifically requires that the Secretary of Transportation establish a program requiring States to conduct life-cycle costs analysis on NHS projects where the cost of a usable project segment equals or exceeds $25 million. The FHWA’s Executive Director, Anthony Kane, distributed implementing guidance on NHS LCCA requirements to FHWA field offices in an April 19, 1996, memorandum. The implementing guidance focused on the use of *good practice* rather than prescribe specific LCCA procedures.

The NHS Designation Act of 1995 also required the SHAs to perform Value Engineering Analysis on the same high-cost NHS projects. The Value Engineering provisions were implemented in 23 Code of Federal Regulations (CFR) Part 627 published in the *Federal Register* in February 1997, and requirements took effect March 17, 1997.(2)

Finally, the 1998 Transportation Equity Act for the 21st Century (TEA-21) removed the LCCA requirements established in the NHS Act and directed the Secretary of Transportation to develop recommended procedures for conducting LCCA on NHS projects. Such recommended procedures are to be developed in consultation with AASHTO and in concert with the principles defined in Executive Order 12893.

GENERAL DEFINITIONS

Some of the more general definitions used in this *Technical Bulletin* are listed below. Other definitions are provided in the sections where they are addressed.

Life-Cycle Cost Analysis (LCCA), was legislatively defined in Section 303, *Quality Improvement*, of the National Highway System NHS Designation Act of 1995. The definition as modified by TEA-21, is "... a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, user, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment." A ***usable project*** segment is defined as a portion of a highway that, when completed, could be opened to traffic independent of some larger overall project.

In simpler terms, LCCA is an analysis technique that supports more informed and, it is hoped, better investment decisions. It builds on some well-founded principles of economic analysis that have been used to evaluate highway and other public works investments for years, but LCCA has a slightly stronger focus on the longer term. It incorporates discounted long-term agency, user, and other relevant costs over the life of a highway or bridge to identify the best value for investment expenditures (i.e., the lowest long-term cost that satisfies the performance objective sought). LCCA can be applied to a wide variety of investment-related decision levels to evaluate the economic worth of various designs, projects, alternatives, or system investment strategies to get the best return on the dollar.

Pavement Design is defined under 23 CFR Section 500.203 as "... a project-level activity where detailed engineering and economic considerations are given to alternate combinations of subbase, base, and surface material which will provide adequate load carrying capacity. Factors that are considered include: materials, traffic, climate, maintenance, drainage and life cycle costs."

User Costs are costs incurred by highway users traveling on the facility and the excess costs incurred by those who cannot use the facility because of either agency or self-imposed detour requirements. User costs typically are an aggregation of three separate components: Vehicle Operating Costs (VOC), Crash Costs, and User Delay Costs. Chapter 3 discusses each of these cost components in detail.

Deterministic Approach to LCCA applies procedures and techniques without regard for the variability of the inputs. The primary disadvantage of this traditional approach is that it does not account for the variability associated with the LCCA input parameters.

Risk Analysis Approach characterizes uncertainty. This *Interim Technical Bulletin* advocates this approach because it combines probability descriptions of analysis inputs with computer simulations to generate the entire range of outcomes as well as the likelihood of occurrence.

Economic Indicators

Several ***economic indicators*** are available to the analyst. The most common include Benefit/Cost (B/C) Ratios, Internal Rate of Return (IRR), Net Present Value (NPV), and Equivalent Uniform Annual Costs (EUAC). Many of these indicators are thoroughly discussed in the 1992 Office of Management and Budget Circular A-94.(3)

Benefit/Cost Analysis or Ratio represents the net discounted benefits of an alternative divided by net discounted costs. B/C ratios greater than 1.0 indicate that benefits exceed cost. The B/C ratio approach is generally not recommended for pavement analysis because of the difficulty in sorting out benefits and costs for use in developing B/C ratios.

Internal Rate of Return, primarily used in private industry, represents the discount rate necessary to make discounted cost and benefits equal. While the IRR does not generally provide an acceptable decision criterion, it does provide useful information, particularly when budgets are constrained or there is uncertainty about the appropriate discount rate.

Net Present Value, sometimes called *Net Present Worth (NPW)*, is the discounted monetary value of expected net benefits (i.e., benefits minus costs). NPV is computed by assigning monetary values to benefits and costs, discounting future benefits ($PV_{benefits}$) and costs (PV_{costs}) using an appropriate discount rate, and subtracting the sum total of discounted costs from the sum total of discounted benefits.

Discounting benefits and costs transforms gains and losses occurring in different time periods to a common unit of measurement. Programs with positive NPV value increase social resources and are generally preferred. Programs with negative NPV should generally be avoided. There is fairly strong agreement in the literature that NPV is the economic efficiency indicator of choice.
The basic formula for computing NPV is:

$$\mathrm{NPV} = \mathrm{PV}_{\mathrm{benefits}} - \mathrm{PV}_{\mathrm{costs}}$$

Because the benefits of keeping the roadway above some preestablished terminal service ability level are the same for all design alternatives, the benefits component drops out and the formula reduces to:

$$NPV = \text{Initial Cost} + \sum_{k=1}^{N} \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right]$$

where: i = discount rate
n = year of expenditure

The section on *Compute Net Present Value* (page 25) discusses NPV computations in more detail.

Equivalent Uniform Annual Costs represents the NPV of all discounted cost and benefits of an alternative as if they were to occur uniformly throughout the analysis period. EUAC is a particularly useful indicator when budgets are established on an annual basis. The preferred method of determining EUAC is first to determine the NPV, and then use the following formula to convert it to EUAC:

$$EUAC = NPV\left[\frac{1(1+i)^n}{(1+i)^n - 1}\right]$$

where: i = discount rate
n = number of years into future

Additional terms are defined as necessary as they occur in the body of the text.

COST ESTIMATES

Estimates of future costs and benefits can be made using *constant* or *nominal* dollars. *Constant* dollars, often called *real* dollars, reflect dollars with the same or constant purchasing power over time. In such cases, the cost of performing an activity would not change as a function of the future year in which it would be accomplished. For example, if hot-mix asphalt concrete (HMAC) costs $20/ton today, then $20/ton should be used for future year HMAC cost estimates. *Nominal* dollars, on the other hand, reflect dollars that fluctuate in purchasing power as a function of time. They are normally used to fold in future general price rises resulting from anticipated inflation. When using *nominal* dollars, the estimated cost of an activity would change as a function of the future year in which it is accomplished. In this case, if HMAC costs $20/ton today, and inflation were estimated at 5 percent, HMAC cost estimates for 1 year from today would be $21/ton.

While LCCA can be conducted using either *constant* or *nominal* dollars, there are two cautions. First, in any given LCCA, *constant* and *nominal* dollars cannot be mixed in the same analysis (i.e., all costs must be in either *constant* dollars or all costs must be in *nominal* dollars). Second, the discount rate (discussed below) selected must be consistent with the dollar type used (i.e., use *constant* dollars and discount rates or *nominal* dollars and discount rates). *Good practice* suggests conducting LCCA using *constant* dollars and *real* discount rates. This combination eliminates the need to estimate and include an inflation premium for both cost and discount rates.

DISCOUNT RATES

Nominal Versus Real

Similar to costs, LCCA can use either real or nominal discount rates. *Real* discount rates reflect the true time value of money with no inflation premium and should be used in conjunction with noninflated dollar cost estimates of future investments. *Nominal* discount rates include an inflation component and should only be used in conjunction with inflated future dollar cost estimates of future investments. The same caveats, as noted above, apply to mixing real dollar cost and nominal discount rates and vice versa. The OMB Circular A-94, and the annual updates of appendix A to the Circular, further discuss the *real versus nominal* dollar and discount rates issue.

Values to Use

Discount rates can significantly influence the analysis result. LCCA should use a reasonable discount rate that reflects historical trends over long periods of time. Data on the historical trends over very long periods indicate that the real time value of money is approximately 4 percent.

In the public sector, because investment resources come from Jane and John Q. Public in the form of taxes or user fees, the discount rate used needs to be consistent with the opportunity cost of the public at large. The supersafe U.S. Government Treasury Bill is one conservative indicator of the opportunity cost of money for the public at large. Figure 1.1 reflects the historical trend of yields on 10-year Treasury notes. The upper curve reflects the nominal rate of return while the lower curve represents the inflation adjusted real rate of return. For the period March 1991 through August 1996, the real rate of return ranges somewhere between 3- to 5-percent and the average close to 4 percent.

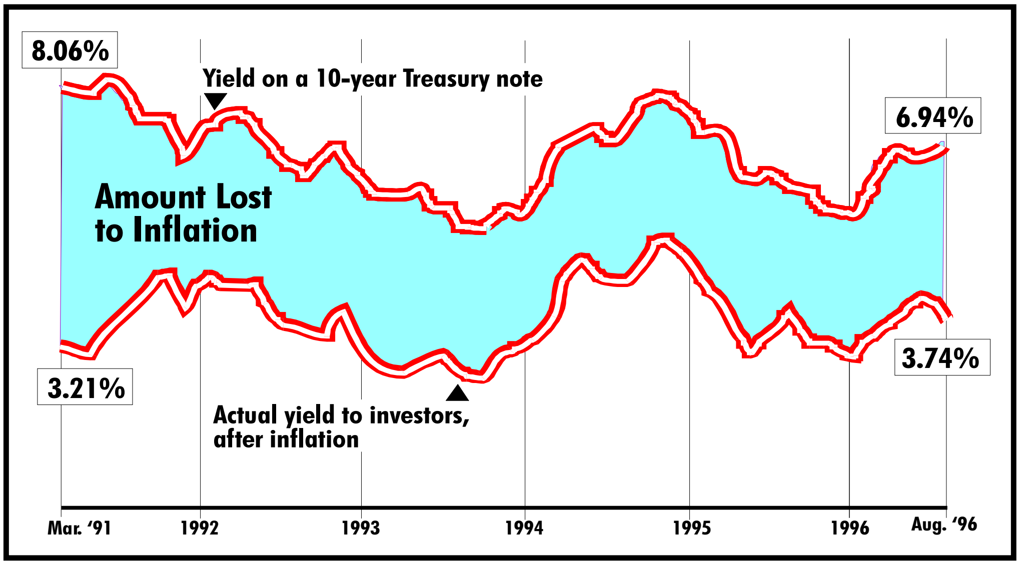


Figure 1.1. Historical trends on 10-year Treasury notes.

The Department of the Treasury made its first offering of Inflation Protected Securities to the general public in spring 1997. These securities offer a real rate of return and a provision that adjusts the principal to protect against inflation. The offering was very well received by the public (there was more demand for the securities than the Treasury Department wanted to sell) at a yield of just over 3.5 percent.

In 1995 and 1996, the FHWA Office of Engineering, Pavement Division, conducted a national pavement design review and found that the discount rates currently employed by SHAs to conduct LCCA in pavement design showed a distribution of values clustering in the 3- to 5-percent range.

Finally, table 1.1 shows recent trends in real discount rates for various analysis periods published over the last several years in annual updates to OMB Circular A-94. Considering the above, *good practice* suggests using a real discount rate, one that does not reflect an inflation premium, of 3 to 5 percent in conjunction with real/constant dollar cost estimates.

Table 1.1. Recent trends in OMB real discount rates.

Year	**Analysis Period**				
	3	**5**	**7**	**10**	**30**
92	2.7	3.1	3.3	3.6	3.8
93	3.1	3.6	4.0	4.3	4.5
94	2.1	2.3	2.5	2.7	2.8
95	4.2	4.5	4.6	4.8	4.9
96	2.7	2.7	2.8	2.8	3.0
97	3.2	3.3	3.4	3.5	3.6
98	3.4	3.5	3.5	3.6	3.8
Average	3.1	3.3	3.4	3.6	3.8
Standard Deviation	0.7	0.7	0.7	0.8	0.7

STRUCTURED APPROACH

Analysts should work from formalized, objective LCCA procedures incorporated within the overall pavement design process. Such procedures should be comprehensive enough to capture and evaluate the differences between competing pavement design alternatives and subsequent rehabilitation strategies. The design process should clearly identify when and at what level to perform the LCCA, as well as the scope and level of detail of such analysis. LCCA procedures should clearly identify the components and factors that are included in addition to supporting rationale for selected input values. LCCA input assumptions should be reasonable and conform to accepted practice and convention. LCCA should recognize the uncertainty associated with LCCA inputs and the implication of the uncertainty on LCCA results. As a minimum, LCCA should include a sensitivity analysis of LCCA results to variation in major LCCA inputs. SHAs are encouraged to incorporate a quantitative risk analysis approach to treat input uncertainty (see chapter 4).

CHAPTER 2. LCCA PROCEDURES

This chapter identifies the procedural steps involved in conducting a life-cycle cost analysis (LCCA). They include:

1. Establish alternative pavement design strategies for the analysis period.
2. Determine performance periods and activity timing.
3. Estimate agency costs.
4. Estimate user costs.
5. Develop expenditure stream diagrams.
6. Compute net present value.
7. Analyze results.
8. Reevaluate design strategies.

While the *steps* are generally sequential, the sequence can be altered to meet specific LCCA needs. The following sections discuss each step.

ESTABLISH ALTERNATIVE PAVEMENT DESIGN STRATEGIES FOR THE ANALYSIS PERIOD

The primary purpose of an LCCA is to quantify the long-term implication of initial pavement design decisions on the future cost of maintenance and rehabilitation activities necessary to maintain some preestablished minimum acceptable level of service for some specified time.

A *Pavement Design Strategy* is the combination of initial pavement design and necessary supporting maintenance and rehabilitation activities. *Analysis Period* is the time horizon over which future cost are evaluated. The first step in conducting an LCCA of alternative pavement designs is to identify the alternative pavement design strategies for the analysis period under consideration.

Analysis Period

LCCA analysis period should be sufficiently long to reflect long-term cost differences associated with reasonable design strategies. The analysis period should generally always be longer than the pavement design period, except in the case of extremely long-lived pavements. As a rule of thumb, the analysis period should be long enough to incorporate at least one rehabilitation activity. The FHWA's September 1996 Final LCCA Policy statement recommends an analysis period of *at least* 35 years for all pavement projects, including new or total reconstruction projects as well as rehabilitation, restoration, and resurfacing projects.(4)

At times, a shorter analysis periods may be appropriate, particularly when pavement design alternatives are developed to buy time (say 10 years) until total reconstruction. It may be appropriate to deviate from the recommended minimum 35-year analysis period when slightly shorter periods could simplify salvage value computations. For example, if all alternative strategies would reach terminal serviceability at year 32, then a 32-year analysis would be quite appropriate.

Regardless of the analysis period selected, the analysis period used should be the same for all alternatives. Figure 2.1 shows a typical analysis period for a pavement design alternative.

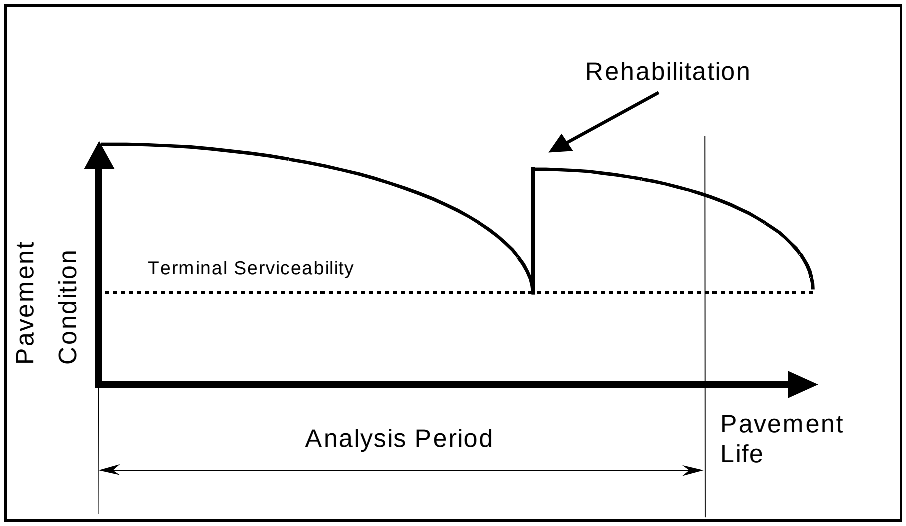


Figure 2.1. Analysis period for a pavement design alternative.

Pavement Design Strategies

Typically, each design alternative will have an expected initial design life, periodic maintenance treatments, and possibly a series of rehabilitation activities. It is important to identify the scope, timing, and cost of these activities. Depending on the initial pavement design, SHAs employ a variety of rehabilitation strategies to keep the highway facilities in functional condition.(5,6) For example, table 2.1 shows the Pennsylvania Department of Transportation's (PennDOT's) typical supporting maintenance and rehabilitation strategy for new, reconstructed, and unbonded portland cement concrete pavements included in its LCCA procedures. PennDOT's LCCA procedures also contain typical supporting strategies for new and reconstructed asphalt concrete pavements. Note that user cost requirements are also identified.

DETERMINE PERFORMANCE PERIODS AND ACTIVITY TIMING

Performance life for the initial pavement design and subsequent rehabilitation activities has a major impact on LCCA results. It directly affects the frequency of agency intervention on the highway facility, which in turn affects agency cost as well as user costs during periods of construction and maintenance activities. SHAs can determine specific performance information for various pavement strategies through analysis of pavement management data and historical experience. Operational pavement management systems can provide the data and analysis techniques to evaluate pavement condition and performance and traffic volumes to identify cost-effective strategies for short- and long-term capital projects and maintenance programs. Some SHAs develop performance lives based on the collective experience of their senior engineers.

Table 2.1. PennDOT's design strategy for new, reconstructed, and unbonded overlay.

Year	Treatment
5	Clean and seal 25% of longitudinal joints. Clean and seal 5% of transverse joints. 0% for neoprene seals. Seal coat shoulders if Type 1 paved shoulders.
10	Same as year 5.
15	Clean and seal 25% of longitudinal joints. Clean and seal 10% of transverse joints, 5% for neoprene seals. Seal coat shoulders, if Type 1 paved shoulders.
20	Concrete patch 5% of pavement area Spall repair 1% of transverse joints (5 sf/joint). Slab stabilization: minimum 25% of transverse joint. Diamond grind 100% of pavement area. Clean and seal all longitudinal joints, including shoulders. Clean and seal all transverse joints, 7% for neoprene seals. Seal coat shoulders, if Type I paved shoulders. Maintenance and protection of traffic. User delay.
25	Clean and seal 25% of longitudinal joints. Clean and seal 10% of transverse joints, 10% for neoprene seals Seal coat shoulders, if Type I paved shoulders.
30	Concrete patch 2% of pavement area. Clean and seal all joints with fiber asphalt membrane. 60-#/sy leveling course. 3.5-in ID-2 or 4-in ID-3/ID-2 overlay. Saw and seal joints. Type 7 paved shoulders. Adjust all guide rail and drainage structures. Maintenance and protection of traffic. User delay.
35	Seal coat shoulders.

Note: The CPR strategy slated for year 20 can be moved to year 15 at the District's discretion. However, when doing this, the overlay at year 30 must be moved to year 25, and another overlay added at year 33.

Current FHWA efforts to analyze pavement performance data collected as part of the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance Program (LTPP) should provide an additional valuable resource to SHAs. To support that effort, the FHWA is also coordinating the development and wide distribution of the DataPave software program to make LTPP performance data directly available to the SHAs. Specific pavement performance information is also available in various pavement performance reports developed by SHAs such as Minnesota and Illinois, just to mention a few.

Work zone requirements for initial construction, maintenance, and rehabilitation directly affect highway user costs and should be estimated along with pavement strategy development. The frequency, duration, severity, and year of work zone requirement are critical factors in developing user costs for the alternatives being considered.

ESTIMATE AGENCY COSTS

Construction quantities and costs are directly related to the initial design and subsequent rehabilitation strategy. The first step in estimating agency costs is to determine construction quantities/unit prices. Unit prices can be determined from SHA historical data on previously bid jobs of comparable scale. Other data sources include the Bid Analysis Management System (BAMS), if used by the SHA.

LCCA comparisons are always made between mutually exclusive competing alternatives. LCCA need only consider differential costs between alternatives. Costs common to all alternatives cancel out, these cost factors are generally noted and excluded from LCCA calculations.

Agency costs include all costs incurred directly by the agency over the life of the project. They typically include initial preliminary engineering, contract administration, construction supervision and construction costs, as well as future routine and preventive maintenance, resurfacing and rehabilitation cost, and the associated administrative cost. Routine reactive-type maintenance cost data are normally not available except on a very general, areawide cost per lane mile. Fortunately, routine reactive-type maintenance costs generally are not very high, primarily because of the relatively high performance levels maintained on major highway facilities. Further, SHAs that do report routine reactive-type maintenance costs note little difference between most alternative pavement strategies. When discounted to the present, small reactive maintenance cost differences have negligible effect on NPV and can generally be ignored.

Agency costs also include maintenance of traffic cost and can include operating cost such as pump station energy costs, tunnel lighting, and ventilation. At times, the salvage value, the remaining value of the investment at the end of the analysis period, is included as a negative cost.

Salvage Value represents value of an investment alternative at the end of the analysis period. The two fundamental components associated with salvage value are residual value and serviceable life.

Residual Value refers to the net value from recycling the pavement. The differential residual value between pavement design strategies is generally not very large, and, when discounted over 35 years, tends to have little effect on LCCA results.

Serviceable Life represents the more significant salvage value component and is the remaining life in a pavement alternative at the end of the analysis period. It is primarily used to account for differences in remaining pavement life between alternative pavement design strategies at the end of the analysis period. For example, over a 35-year analysis, Alternative A reaches terminal serviceability at year 35, while Alternative B requires a 10-year design rehabilitation at year 30. In this case, the serviceable life of Alternative A at year 35 would be 0, as it has reached its terminal serviceability. Conversely, Alternative B receives a 10-year design rehabilitation at year

30 and will have 5 years of serviceable life at year 35, the year the analysis terminates. The value of the serviceable life of Alternative B at year 35 could be calculated as a percent of design life remaining at the end of the analysis period (5 of 10 years or 50 percent) multiplied by the cost of Alternative B's rehabilitation at year 30.

Sunk Costs represent a special category of costs that are irrelevant to the decision at hand. Analysts should be careful not to include them in LCCA. An example may serve best in understanding the concept.

> *An individual places a $10 nonrefundable deposit on a $100 camera at Store A. Before picking up the camera, the individual finds an identical camera on sale at Store B for $80. From an economic efficiency perspective, from which store should the individual purchase the camera? What bearing does the $10 deposit have on the decision?*

The $10 deposit is a sunk cost and is irrelevant to the decision. The decision comes down to paying Store A the $90 balance for the camera, or paying Store B $80 for an identical camera.

Not all cases of sunk cost are this clear and, again, analysts need to take care to guard against including them in LCCA. An example more specific to pavement design might involve the reluctance of a designer to select an alternative with a much lower life-cycle cost because it would mean *wasting* the money previously spent on developing final plans for a clearly inferior alternative.

ESTIMATE USER COSTS

In the simplest sense, user costs are costs incurred by the highway user over the life of the project. In LCCA, highway user costs of concern are the *differential* costs incurred by the motoring public between competing alternative highway improvements and associated maintenance and rehabilitation strategies over the analysis period. In the pavement design arena, the user costs of interest are further limited to the differences in user costs resulting from differences in long-term pavement design decisions and the supporting maintenance and rehabilitation implications. User costs are an aggregation of three separate cost components: vehicle operating costs (VOC), user delay costs, and crash costs.

Normal Operations Versus Work Zones

In the LCCA of pavement design alternatives, there are user costs associated with both normal operations and work zone operations. The *normal operations category* reflects highway user costs associated with using a facility during periods free of construction, maintenance, and/or rehabilitation (i.e., work zone) activities that restrict the capacity of the facility. User costs in this category are a function of the differential pavement performance (roughness) of the alternatives. The *work zone operations category*, however, reflects highway user costs associated with using a facility during periods of construction, maintenance, and/or rehabilitation activities that generally restrict the capacity of the facility and disrupt normal traffic flow.

During normal operating conditions, as a general rule, there should be little difference between crash costs and delay costs resulting from pavement design decisions. Further, as long as the pavement performance levels remain relatively high, and performance curves of the alternative designs are similar, there should be little if any difference between vehicle operating costs.

If, however, pavement performance curves and levels differ substantially, significant vehicle operating cost differentials can develop. Figure 2.2 depicts an exaggerated example of alternative pavement design strategies.

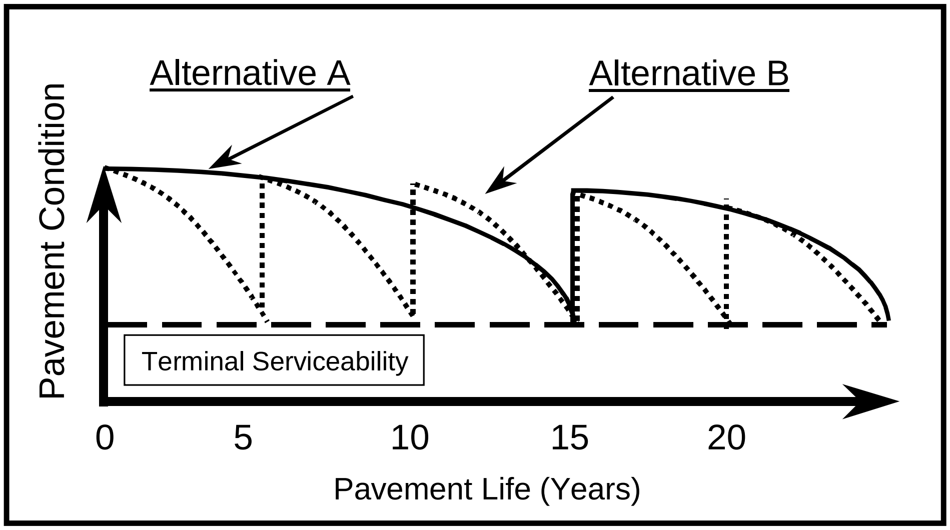


Figure 2.2. Performance curve versus rehabilitation strategy.

In figure 2.2, Alternative A represents a traditional longer term strategy with rehabilitation implemented on a 15-year cycle. Alternative B consists of a minimal treatment on a 5-year cycle. This figure graphically depicts differences in performance levels for different rehabilitation strategies. Intuitively, differences in pavement performance can produce differences in vehicle operating costs. Slight differences in VOC rates caused by differences in pavement performance characteristics (primarily roughness), when multiplied by several years Vehicle Mile(s) Traveled (VMT), could result in huge VOC differentials over the life of the design strategy. This is particularly true for pavement preservation strategies that exhibit poor performance over most of the analysis period as shown by Alternative B in figure 2.2.

To calculate these differences, however, the analysis must be able to:

(1) Accurately estimate the pavement performance differences over time (at least yearly).
(2) Quantify the difference in VOC rates for slight differences in pavement performance at relatively high performance levels.

Most research on VOC rates, as a function of pavement performance, has been conducted by the World Bank. Figure 2.3 shows the effect of road roughness, as measured by the international roughness index (IRI), on road user costs in New Zealand.(7) The figure also shows that additional operating costs (as compared to a smooth road baseline) begin to accrue around an IRI equal to 170 in/mi. According to Darter's and Al-Omai's work, an IRI level of 170 is approximately equal to a PRS rating of 2.5. On higher order systems in the United States, such as the NHS, SHAs typically consider pavements with a PSR of 2.5 to have reached their terminal serviceability index and schedule some form of rehabilitation.(8)

The effect of pavement condition on user operating cost at low roughness levels, if any, is not well documented. There is, however, current research, NCHRP 1-33, *Methodology to Improve Pavement Investment Decision*, under way in this area. NCHRP 1-33 has been initiated to obtain objective information relating costs associating with truck operating expenses (health claims, cargo damage, vehicle depreciation, maintenance and repair) with road roughness. This study is scheduled to be complete by 1999.

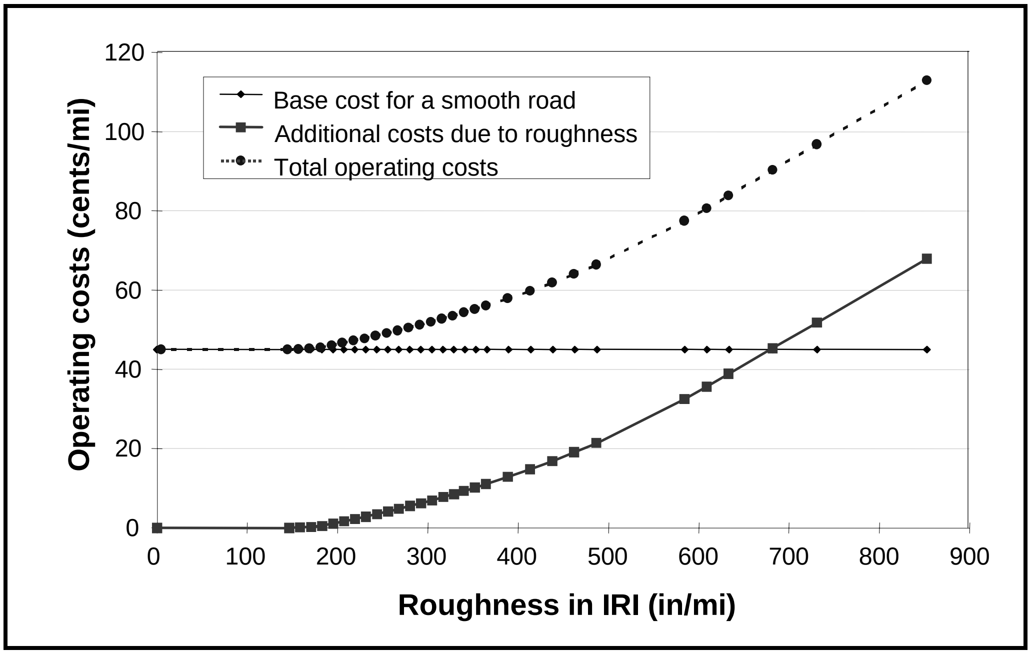


Figure 2. 3. Effect of roughness on road user costs in New Zealand.

Additionally, the Cornell University School of Civil Engineering is performing preliminary work on establishing differential vehicle operating costs associated with pavement condition (i.e., IRI) for the New York State Department of Transportation.(9)

Even if user operating cost differentials are established between smooth and very smooth roads, the analyst must still overcome the difficulty in estimating projected year-by-year performance differences between alternative pavement design strategies.

Considering the prevailing pavement performance ranges encountered in the United States on higher type facilities, and the lack of precision in projecting year-by-year pavement performance differentials, this *Interim Technical Bulletin* does not address computing user vehicle operating cost differentials during normal operating conditions at this time.

On the other hand, during periods of initial construction and future maintenance and rehabilitation activities (i.e., work zone operation), vehicle operating cost, user delay, and crash costs can be significantly different between alternative pavement design strategies. As a result, this *Technical Bulletin* focuses primarily on work zone user costs.

User Cost Rates

User cost rates, as used here, refer to the dollar values assigned to each user cost component. User costs are calculated by multiplying the quantity of the various additional user cost components (VOC, delay, and crash) incurred by the unit cost for those cost components.

Additional VOCs are determined by multiplying the quantity of additional VOC components (i.e., additional speed changes, stops, miles, hours of idling) incurred by the dollar value assigned to each VOC component. By the same token, user delay costs are determined by multiplying the additional hours of travel time resulting from WZ-caused traffic delay (or additional miles of travel caused by detours) by the dollar value of an hour of delay for each vehicle classification. Finally, the additional crash costs are determined by multiplying the number of additional crashes (by type) by the appropriate dollar value assigned to each crash type.

Chapter 3 presents detailed procedures to calculate work zone user cost quantities of alternate pavement design strategies, while the unit costs associated with each cost component are discussed below.

VOC Rates

Table 5 of NCHRP Report 133, *Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects,* can be used to determine VOC rates for stopping/speed changes and idling, as well as associated delay times for stopping/speed changes. This work is based on the earlier work by Winfrey, *Economic Analysis for Highways*.(10,11) A compressed version of NCHRP 133 table 5 is reproduced as table 2.2.

Table 2.2 shows additional hours of delay and additional VOC associated with stopping 1,000 vehicles from a particular speed and returning them to that speed. Different factors are provided for passenger cars and single-unit and combination trucks. In addition, the table includes a vehicle operating cost associated with idling while stopped. The cost factors reflect 1970 prices based on a $3 per hour value of time for passenger vehicles and $5 per hour for all trucks.

To make these factors applicable to current analysis, the values shown have been escalated to reflect more current year dollars. The escalation factor for VOC is determined by using the transportation component of the Consumer Price Index (CPI) for the base year (1970) and the more current year (August 1996). The transportation component of the CPI was 37.5 in 1970,

Table 2.2. Added time and vehicle running cost/1,000 stops and idling costs (1970 $).

Initial Speed (mi/h)	Added Time (Hr/1,000 Stops) (Excludes Idling Time)			Added Cost ($/1,000 Stops) (Excludes Idling Time)		
	Pass Cars	Single-Unit Truck	Combination Truck	Pass Cars	Single-Unit Truck	Combination Truck
5	1.02	0.73	1.10	0.71	2.43	8.83
10	1.51	1.47	2.27	2.32	5.44	20.35
15	2.00	2.20	3.48	3.98	8.90	34.13
20	2.49	2.93	4.76	5.71	12.71	49.91
25	2.98	3.67	6.10	7.53	16.80	67.37
30	3.46	4.40	7.56	9.48	21.07	86.19
35	3.94	5.13	9.19	11.57	25.44	106.05
40	4.42	5.87	11.09	13.84	29.93	126.63
45	4.90	6.60	13.39	16.30	34.16	147.62
50	5.37	7.33	16.37	18.99	38.33	168.70
55	5.84	8.07	20.72	21.92	42.25	189.54
60	6.31	8.80	27.94	25.13	47.00	209.82
65	6.78	9.53	NA	28.63	51.43	NA
70	7.25	NA	NA	32.46	NA	NA
75	7.71	NA	NA	36.64	NA	NA
80	8.17	NA	NA	41.19	NA	NA
Idling Cost ($/Veh-Hr)				0.1819	0.2017	0.2166

Source: R. Winfrey, *Economic Analysis for Highways,* and table 5, NCHRP Report 133.
Added Cost ($/1,000 Stops) includes fuel, tires, engine oil, maintenance, and depreciation *Idling Cost ($/Veh-Hr)* includes fuel, engine oil, maintenance, and depreciation.

and 142.8 in August 1996. The VOC escalation factor used to escalate 1970 prices to August 1996 prices is:

$$\underset{\text{(VOC)}}{\text{Escalation Factor}} = \frac{142.8\ \text{(Aug 1996)}}{37.5\ \ \text{(1970)}} = 3.808$$

The *Added Cost per 1,000 Stops* columns in table 2.3, as well as the *Idling Cost*, reflect the adjusted values using the above 3.808 index to establish new August 1996 base year prices.

This table is designed to determine stopping cost, but it can also be used to determine the speed change cost, which is additional cost (VOC and delay) of slowing from one speed to another and returning to the original speed. Speed change costs are calculated by subtracting the cost and time factors of stopping at one speed from the cost and time factors of stopping at another speed. For example, table 2.4 shows the speed change costs of going from 55 mi/h to 40 mi/h and back to 55 mi/h.

Mileage Rates

In addition to these incremental VOCs associated with changes from normal operating condition, there is also a fundamental overall baseline VOC mileage rate associated with normal

Table 2.3. Added time and vehicle running cost/1,000 stops and idling costs (Aug 96 $).

Initial Speed (mi/h)	Added Time (Hr/1,000 Stops) (Excludes Idling Time)			Added Cost ($/1,000 Stops) (Excludes Idling Time)		
	Pass Cars	Single-Unit Truck	Combination Truck	Pass Cars	Single-Unit Truck	Combination Truck
5	1.02	0.73	1.10	2.70	9.25	33.62
10	1.51	1.47	2.27	8.83	20.72	77.49
15	2.00	2.20	3.48	15.16	33.89	129.97
20	2.49	2.93	4.76	21.74	48.40	190.06
25	2.98	3.67	6.10	28.67	63.97	256.54
30	3.46	4.40	7.56	36.10	80.23	328.21
35	3.94	5.13	9.19	44.06	96.88	403.84
40	4.42	5.87	11.09	52.70	113.97	482.21
45	4.90	6.60	13.39	62.07	130.08	562.14
50	5.37	7.33	16.37	72.31	145.96	642.41
55	5.84	8.07	20.72	83.47	160.89	721.77
60	6.31	8.80	27.94	95.70	178.98	798.99
65	6.78	9.53	NA*	109.02	195.84	NA*
70	7.25	NA*	NA*	123.61	NA*	NA*
75	7.71	NA*	NA*	139.53	NA*	NA*
80	8.17	NA*	NA*	156.85	NA*	NA*
Idling Cost ($/Veh-Hr)				0.6927	0.7681	0.8248

*Original data did not provide values for trucks at higher speed. Analysts will need to extrapolate these values when truck calculations are needed at these higher speeds.

Table 2.4. Speed change computations.

Initial Speed (mi/h)	Added Time (Hr/1,000 Stops) (Excludes Idling Time)			Added Cost ($/1,000 Stops) (Excludes Idling Time)		
	Pass Cars	Single-Unit Truck	Combination Truck	Pass Cars	Single-Unit Truck	Combination Truck
55	5.84	8.07	20.72	83.47	160.89	721.77
40	4.42	5.87	11.09	52.70	113.97	482.21
55-40-55	1.42	2.20	9.63	30.77	46.92	239.56

operating conditions. Typically this is expressed as an overall cents-per-mile rate. These rates would typically apply to any additional miles that must be driven because of detours.

Some readily apparent values are the marginal cost rates used by the Federal Government. Federal travel regulations authorize the payment of $0.31 per mile for using privately owned passenger vehicles for official government travel. The flat mileage rate allowed by the IRS for business use of a privately owned passenger vehicle is also $0.31 per mile (tax year 1996).

Delay Cost Rates (Value of Time)

Of all of the user costs rates, the cost rate assigned to user delay (i.e., the value of time) is by far the most controversial. As a result, the cost rates for user delay are discussed from several perspectives. The sources used in this *Interim Technical Bulletin* include updated values from earlier NCHRP research, recent guidance provided by the Office of the Secretary of Transportation (OST), and recently updated values used by the FHWA in its Highway Economic Requirements System (HERS) Model.

Earlier Research Studies

A base case value can be generated from the earlier NCHRP 133 report, which used 1970 dollar values of $3 per hour passenger vehicles and $5 per hour for all trucks. Once again, these values must be escalated to reflect more current/base year values.

In this case, the escalation factor for the dollar value of time is determined by using changes to the *All Items Component* of the CPI for the base year (1970) and the more current year (August 1996). The *All Items Component* of the CPI was 38.8 in 1970 and 152.4 in August 1996. The value of time escalation factor to escalate 1970 prices to August 1996 prices is:

$$\begin{array}{c}\text{Escalation Factor}\\ \text{(Value of Time)}\end{array} = \frac{152.4\ \text{(Aug 1996)}}{38.8\ \text{(1970)}} = 3.928$$

Table 2.5 shows the updated value of time.

Table 2.5. Updated NCHRP 133 values of time ($/Veh-Hr) (Aug 96 $).

Value of Time	Pass Cars	Trucks	
		Single-Unit	Combination
Value 1970 Factor 8/96	$3.00 3.928	$5.00 3.928	$5.00 3.928
Value 8/96	11.78	19.64	19.64

Table 2.6 shows more recent work (1993) from the computerized MicroBENCOST program developed under NCHRP Research Project 7-12, *Microcomputer Evaluation of Highway User Benefits*, which uses 1990 base year default values. The table includes another escalation factor to bring these 1990 base year costs to a new August 1996 base year. Once again, the escalation factor for the value of time is calculated by dividing the August 1996 overall CPI (152.4) by the overall CPI for 1990 (130.7).

$$\underset{\text{(Aug 1996)}}{\text{Escalation Factor}} = \frac{152.4 \quad \text{(Aug 1996)}}{130.7 \quad \text{(1990)}} = 1.166$$

Table 2.6. Updated MicroBENCOST default values of time ($/Veh-Hr) (Aug 96 $).

Value of Time	Pass Cars	Single-Unit Trucks		Combination Trucks	
		2 AX 12 Kips	3 AX 35 Kips	2S2 40 Kips	3S2 63 Kips
Value 1990	$9.75	$13.64	$16.28	$20.30	$22.53
		$14.96		$21.42	
Factor 8/96	1.166	1.166		1.166	
Value 8/96	$11.37	$17.44		$24.98	

Table 2.7 brings together the updated values from both the earlier NCHRP 133 and MicroBENCOST reports and shows the average values per vehicle hour for the three vehicle classifications in August 1996 dollars. Prior to using these values the analysts should escalate their values to reflect current costs using a similar approach.

Table 2.7. Composite earlier research value of time ($/Veh-Hr) (Aug 96 $).

Updated Source	Pass Cars	Trucks	
		Single-Unit	Combination
NCHRP 133	$11.78	$19.64	$19.64
MicroBENCOST	11.37	17.44	24.98
Average Value	$11.58	$18.54	$22.31

OST Approach

Recent guidance provided by the U.S. DOT OST to the various DOT modal administrations is another source that can be used to determine the value of time. OST recommends using a percentage of the national wage rate for the value of time. OST-recommended procedures apply different percentages of the national wage rate as a function of vehicle classification and trip type and purpose. Table 2.8 provides U.S. DOT ranges of the percentage of the national wage rate that should be applied to various combinations of trip type and purpose. The rates shown are per person hour. As can be seen in table 2.8, business and truck travel are valued more highly than personal travel, and intercity personal travel is valued more highly than local personal travel.

Table 2.8. Travel time ranges as a percent of national wage rate (1995 $/Person-Hr).

Travel Category	Trip Type	
	Local	Intercity
Personal	35 to 60%	60 to 90%
Business	80 to 120%	80 to 120%
Truck Drivers	100%	100%

U.S. DOT, *The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations.*

Table 2.9 provides information on national hourly wage rates used by OST in 1995 dollars.

Table 2.9. OST-recommended hourly wage rates (1995 $/Person-Hr).

Travel Category	Trip Type	
	Local	Intercity
Personal	$17.00	$17.00
Business	18.80	18.80
Truck Drivers	16.50	16.50

U.S. DOT, *The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations.*

Based on the information on appropriate percentages provided in table 2.8 and the recommended national hourly wage rate provided in table 2.9, the OST developed the recommended ranges for the value of travel time shown in table 2.10. The values associated with *Mixed* are the ranges to be used when the distribution between auto business and personal trips is not known. Hourly values shown for trucks are $16.50 for both local and intercity trip types.

Table 2.10 lists the ranges of values in dollars per person hour. As most user delay analyses compute vehicle hours of delay, these values must be adjusted for vehicle occupancy rates.

Table 2.10. Ranges for hourly values of travel time (1995 $/Person-Hr)

Travel Category	Local		Intercity	
	Low	High	Low	High
Personal	$6.00	$10.20	$10.20	$15.30
Business	15.00	22.60	15.00	22.60
Mixed	6.40	10.70	10.40	15.70
Truck Drivers	16.50	16.50	16.50	16.50

U.S. DOT, *The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations.*

According to the 1990 *Personal Transportation Survey*, typical auto vehicle occupancy rates for personal travel are 1.7 in urban areas and 2.0 in rural areas.(12) The typical vehicle occupancy rates for trucks and auto business use is much closer to 1.0.

FHWA Highway Economic Requirements System (HERS) Model

The FHWA uses the HERS Model in conducting national-level analysis of highway performance, needs, and economic evaluation of proposed highway improvements. Part of the economic analysis includes determining the value of travel time delay. Table 2.11 shows the default dollar values of travel time used in the HERS model.

Table 2.11. Value of one vehicle hour of travel time (1995 $).

Travel Category	**Autos**		**Trucks**				
	Small	**Medium**	**Single-Unit**			**Combinations**	
			4 Tire	**6 Tire**	**3-4 Axle**	**4 Axle**	**5 Axle**
Business	$27.99	$28.29	$20.20	$24.96	$27.02	$31.02	$31.58
Personal	12.78	$12.78	$12.78	NA	NA	NA	NA
% AADT Personal Use	90%	90%	69%	0%	0%	0%	0%
Weighted Average	$14.30	$14.33	$15.08	$24.96	$27.02	$31.02	$31.58

FHWA Highway Economic Requirements System Technical Report v3-1 9/97 (Exhibit 8-7).

The values of travel time shown on lines 1 and 2 of table 2.11 are per vehicle hour based on typical vehicle occupancy rates. The values associated with *% AADT Personal Use* on line 3 represent the percent of travel that is personal. The *Weighted Average* values shown on line 4 are for mixed flow of business and personal travel and are used when traffic flow distribution by travel category is not known. The values shown for trucks are significantly higher than either early research study or the OST guidance. Part of the higher cost (approximately 30 percent) is attributable to value associated with the vehicle cargo and the vehicle itself.

Toll Roads

Finally, another source that can be used to estimate the dollar values of time includes tolls on toll roads in relation to time saved. One of the more interesting revelations of users' willingness to pay comes from the treatment of some High Occupancy Vehicle (HOV) facilities in California and Texas. Both States are experimenting with High Occupancy Toll (HOT) facilities, which allow Single Occupancy Vehicles (SOV) to use an HOV facility for a fee. These studies deal primarily with local auto trips and to some extent may reflect market prices. While the values inferred from such studies tend to indicate lower values of time (approximately $6 per person hour), they may in reality reflect inefficient pricing policy rather than lower values.

Recommended Values of Travel Time (Dollars per Vehicle Hour)

Table 2.12 below is a composite table that brings together the several sources of the value of time previously discussed.

Table 2.12. Composite listing of travel time values.

Source	Units	Autos	Trucks	Combination
U.S. DOT – OST *	$/Person-Hr	$10.80	$16.50	$16.50
MicroBENCOST	$/Veh-Hr	11.37	17.44	24.98
NCHRP	$/Veh-Hr	11.78	19.64	19.64
HERS	$/Veh-Hr	14.30	25.99	31.30

* Values for U.S. DOT — OST reflect dollars per *person* hour

Based on consideration of these potential sources, table 2.13 reflects the ranges of the value of travel time per vehicle recommended for use in typical analyses where distribution data on trip purpose and type are not known.

Table 2.13. Recommended values of time ($/Veh-Hr)(Aug 96 $).

Passenger Cars	Trucks	
	Single-Unit	Combinations
$10 to 13	$17 to 20	$21 to 24

Crash Cost Rates

The MicroBENCOST software package, developed for the NCHRP Research Project 7-12, includes default crash cost rates. Table 2.14 shows the default crash cost rates by crash type for both rural and urban settings in 1990 dollars.

Table 2.14. MicroBENCOST default crash cost rates ($1,000, 1990 $).

Intersection or Facility Type	Fatality		Nonfatal Injury		Property Damage Only (PDO)	
	Rural	Urban	Rural	Urban	Rural	Urban
RR Grade Crossing	$1,008	$994	$25.2	$13.3	$1.59	$3.09
Intersection/Interchange	1,059	932	21.9	14.3	1.98	1.35
Bridge	1,111	978	24.9	14.3	2.14	1.27
Highway Segment	1,111	978	24.9	14.3	2.14	1.27

The MicroBENCOST 1990 dollar default values shown in table 2.14 are escalated to August 1996 dollars in table 2.15 using the 1.116 escalation factor developed earlier.

Table 2.15. MicroBENCOST default crash cost rates ($1,000, Aug 96 $).

Intersection or Facility Type	Fatality		Nonfatal Injury		PDO	
	Rural	Urban	Rural	Urban	Rural	Urban
RR Grade Crossing	$1,125	$1,109	$28.1	$14.8	$1.77	$3.45
Intersection/Interchange	1,182	1,040	24.4	16.0	2.21	1.51
Bridge	1,240	1,091	27.8	16.0	2.39	1.42
Highway Segment	1,240	1,091	27.8	16.0	2.39	1.42

* Values for U.S. DOT-OST reflect dollars per *person* hour

The MicroBENCOST default cost values for a fatality range from $1,091,000 to $1,182,000, which is quite a bit lower than the $2,700,000 cost per fatality averted, which was recommended by the U.S. DOT in its March, 14, 1995 memorandum from the Assistant Secretary for Transportation Policy to DOT Modal Administrators. This $2,700,000 value is an update of the value originally recommended in basic guidance distributed on January 8, 1993.

In addition to the traditional direct work zone user costs, there are indirect user costs such as the impact of user delay on delivery fleet size, the costs associated with a rolling inventory, and other, indirect effects of delay to manufacturing plants that now dependent on *just-in-time* delivery. While such factors will become more important over time, they are beyond the scope of these guidelines. It is interesting to note that cost related to delivery fleet size and the costs associated with a rolling inventory are included in the dollar value of delay time rates used in the FHWA HERS Model.

DEVELOP EXPENDITURE STREAM DIAGRAMS

Expenditure stream diagrams are graphical representations of expenditures over time. They are generally developed for each pavement design strategy to help visualize the extent and timing of expenditures. Figure 2.4. shows a typical expenditure stream diagram.

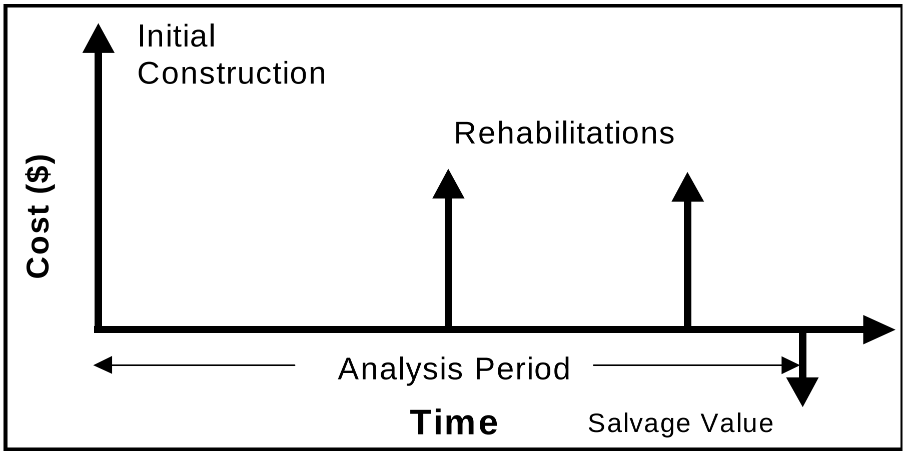


Figure 2.4. Typical expenditure stream diagram for a pavement design alternative.

Normally, costs are depicted as upward arrows at the appropriate time they occur during the analysis period, and benefits are represented as negative cost or downward arrows.

In LCC analysis of pavement design alternatives, the basic benefits of providing and maintaining some preestablished pavement condition level on any given roadway are outside the scope of the analysis. The benefits of providing the specified level of pavement condition are considered to be the same for all pavement design strategies. As a result, the only concerns are the differential costs among alternatives. The only negative cost (i.e., the only downward arrow) would be the cost associated with any salvage value.

Under these conditions, the LCCA objective becomes finding the alternative pavement design strategy that meets the performance requirements at the lowest life-cycle cost.

COMPUTE NET PRESENT VALUE (NPV)

In its broadest sense, LCCA is a form of economic analysis used to evaluate the long-term economic efficiency between alternative investment options. Economic analysis focuses on the relationship between costs, timings of costs, and discount rates employed. Once all costs and their timing have been developed, future costs must be discounted to the base year and added to the initial cost to determine the NPV for the LCCA alternative. As noted earlier, NPV is the economic indicator of choice, and the basic NPV formula for discounting discrete future amounts at various points in time back to some base year is:

$$NPV = \text{Initial Cost} + \sum_{k=1}^{N} \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right]$$

where: i = discount rate and
n = year of expenditure

The $\left[\frac{1}{(1+i)^{n_k}} \right]$ component of the above formula is referred to as the Present Value (PV) factor for a single future amount. PV factors for various combinations of discount rates and future years are available in Discount factor tables (more commonly referred to as interest rate tables). PV for a particular future amount is determined by multiplying the future amount by the appropriate PV factor. Table 2.16 shows discount factors for a single future payment at 3, 4, and 5 percent discount rates for up to 40 years in the future.

Table 2.16. Present value discount factors: single future payment.

	Discount Factor				Discount Factor		
Year	3%	4%	5%	Year	3%	4%	5%
1	0.9709	0.9615	0.9524	21	0.5375	0.4388	0.3589
2	0.9426	0.9246	0.9070	22	0.5219	0.4220	0.3418
3	0.9151	0.8890	0.8638	23	0.5067	0.4057	0.3256
4	0.8885	0.8548	0.8227	24	0.4919	0.3901	0.3101
5	0.8626	0.8219	0.7835	25	0.4776	0.3751	0.2953
6	0.8375	0.7903	0.7462	26	0.4637	0.3607	0.2812
7	0.8131	0.7599	0.7107	27	0.4502	0.3468	0.2678
8	0.7894	0.7307	0.6768	28	0.4371	0.3335	0.2551
9	0.7664	0.7026	0.6446	29	0.4243	0.3207	0.2429
10	0.7441	0.6756	0.6139	30	0.4120	0.3083	0.2314
11	0.7224	0.6496	0.5847	31	0.4000	0.2965	0.2204
12	0.7014	0.6246	0.5568	32	0.3883	0.2851	0.2099
13	0.6810	0.6006	0.5303	33	0.3770	0.2741	0.1999
14	0.6611	0.5775	0.5051	34	0.3660	0.2636	0.1904
15	0.6419	0.5553	0.4810	35	0.3554	0.2534	0.1813
16	0.6232	0.5339	0.4581	36	0.3450	0.2437	0.1727
17	0.6050	0.5134	0.4363	37	0.3350	0.2343	0.1644
18	0.5874	0.4936	0.4155	38	0.3252	0.2253	0.1566
19	0.5703	0.4746	0.3957	39	0.3158	0.2166	0.1491
20	0.5537	0.4564	0.3769	40	0.3066	0.2083	0.1420

NPV Computations

Example NPV computations are provided for the following hypothetical problem. The example is based on a 35-year analysis period.

The initial pavement design will cost $1.1 million and have an associated work zone user cost of $300,000 at year 0. Additional rehabilitation cost of $325,000 will be incurred in years 15 and 30. Associated work zone user costs in years 15 and 30 will be $269,000 and $361,000, respectively. The salvage value at year 35, based on a prorated cost of the year-30 rehabilitation design and remaining life, will be $216,000 (10/15 of $325,000). Figure 2.5 shows the expenditure stream diagram for the example problem.

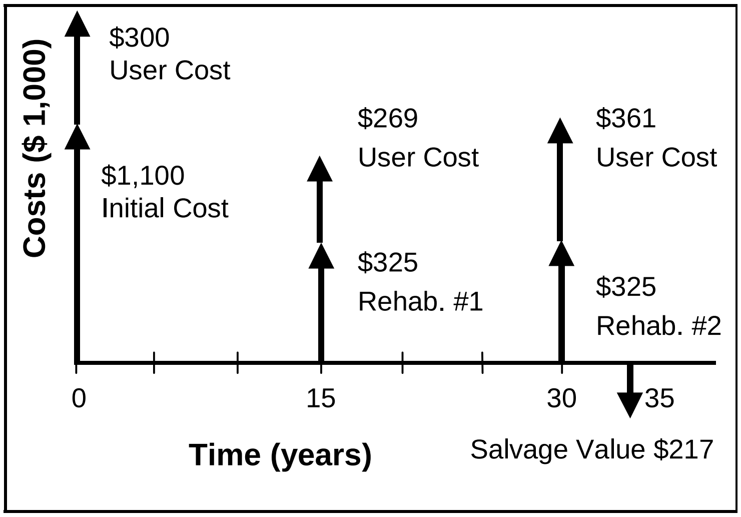


Figure 2.5. Expenditure stream diagram for agency and user costs.

Note that estimated user costs drop in year 15 and go back up in year 30. This is consistent with a longer duration initial work zone followed by short duration rehabilitation work zones impacted by continually increasing traffic volumes over time. Table 2.17 shows the results of PV computations using 4 percent PV factors for single future amounts for the example expenditure stream diagram. The bottom line of table 2.17 shows the NPV of the aggregated individual PVs.

Table 2.17. NPV calculation using 4 percent discount rate factors.

Cost Component Activity	Years	Costs ($1,000)	Discount Factor	Discounted Cost ($1,000)
Initial Construction	0	1,000.0	1.0000	1,000
Initial Work Zone User Cost	0	300.0	1.0000	300
Rehab #1	15	325.0	0.5553	180
Rehab #1 Work Zone User Cost	15	269.0	0.5553	149
Rehab #2	30	325.0	0.3083	100
Rehab #2 Work Zone User Cost	30	361.0	0.3083	111
Salvage Value	35	-216.6	0.2534	-55
Total NPV				1,785

ANALYZE RESULTS

Once completed, all LCCA should, at a minimum, be subjected to a sensitivity analysis. Sensitivity analysis is a technique used to determine the influence of major LCAA input assumptions, projections, and estimates on LCCA results. In a sensitivity analysis, major input values are varied (either within some percentage of the initial value or over a range of values) while all other input values remain constant and the amount of change in results is noted. The input variables may then be ranked according to their effect on results. Sensitivity analysis allows the analyst to subjectively *get a feel for* the impact of the variability of individual inputs on overall LCCA results.

Many times a sensitivity analysis will focus on best case/worst case scenarios in an attempt to bracket outcomes. Most LCC sensitivity analysis, as a minimum, evaluate the influence of the discount rate used on LCCA results.

Sensitivity analyses may be carried out using common spreadsheet-based applications such as Microsoft Excel, Lotus, or Quattro Pro. Tables 2.18 and 2.19 present the results of a spreadsheet analysis of the sensitivity of NPV of two example pavement design strategies to discount rate ranges from 2 to 6 percent for a 35-year analysis period. Total NPV at discount rates ranging from 2 to 6 percent are shown at the bottom of columns (e) through (i).

Table 2.18. Sensitivity analysis – Alternative #1.

Activity (a)	**Year** (b)	**Cost** (c)	**NPV** **2.0%** (e)	**3.0%** (f)	**4.0%** (g)	**5.0%** (h)	**6.0%** (i)
Construction	0	975	975	975	975	975	975
User Cost	0	200	200	200	200	200	200
Rehab #1	10	200	164	149	135	123	112
User Cost # 1	10	269	220	200	182	165	150
Rehab # 2	20	200	135	111	91	75	62
User Cost # 2	20	361	243	200	165	136	113
Rehab # 3	30	200	110	82	62	46	35
User Cost # 3	30	485	268	200	150	1,122	85
Salvage	35	-100	-50	-36	-25	-18	-13
Total NPV			2,266	2,081	1,934	1,815	1,718

Table 2.19. Sensitivity analysis – Alternative #2.

Activity (a)	**Year** (b)	**Cost** (c)	**NPV** **2.0%** (e)	**3.0%** (f)	**4.0%** (g)	**5.0%** (h)	**6.0%** (i)
Construction	0	1,100	1,100	1,100	1,100	1,100	1,100
User Cost	0	300	300	300	300	300	300
Rehab #1	15	325	241	209	180	156	136
User Cost # 1	15	269	200	173	149	129	112
Rehab # 2	30	325	179	134	100	75	57
User Cost # 2	30	361	199	149	111	84	63
Salvage	35	-217	-108	-77	-55	-39	-28
Total NPV			2,112	1,987	1,886	1,805	1,739

Alternative #1 has a lower initial agency cost, and, because of a shorter construction period, a lower user cost than Alternative #2. However, Alternative #1 requires three identical 10-year design rehabilitations compared to two identical 15-year design rehabilitations for Alternative #2.

Out-year user costs for Alternative #1 increase as a result of increased traffic levels over time; out-year user costs for Alternative #2 first decrease due to a shorter work zone period and then increase as a result of increased traffic levels over time.

Both alternatives have a remaining service life at year 35; Alternative #1 has 5 years and Alternative #2 has 10 years. The salvage value, as a prorated share of the last rehabilitation, is 50 percent (5 years remaining on a 10-year design) of its last rehabilitation cost for Alternative #1. For Alternative #1, this translates into 50 percent of the $200,000 year-30 rehabilitation cost. The salvage value of Alternative #2, on the other hand, is 66.6 percent (10 years remaining on a 15-year design) of its last rehabilitation cost. This translates into 66.6 percent of the $325,000 year-30 rehabilitation cost.

Table 2.20 shows a direct comparison of the NPV of both alternatives at several discount rates. Inspection of table 2.20 reveals that the NPV of both alternatives decreases as the discount rate increases. This results from the reduced present value of future costs at higher discount rates. Because the amount and timing of future costs differ between alternatives, the effect of discount rate on NPV is different for each alternative. In this example, Alternative #1 is more expensive than Alternative #2 at discount rates of 5 percent and lower, while Alternative #2 is more expensive than Alternative #1 at discount rates of 6 percent or more. Figure 2.6 shows these results graphically.

Table 2.20. Comparison of alternative NPVs ($1,000) to discount rate.

Activity	**Discount Rate (%)**				
	2.0	**3.0**	**4.0**	**5.0**	**6.0**
Total NPV Alternative #1	2,266	2,081	1,934	1,815	1,718
Total NPV Alternative #2	2,112	1,987	1,886	1,805	1,739
Cost Advantage Alt #2 vs #1	154	94	48	10	-21

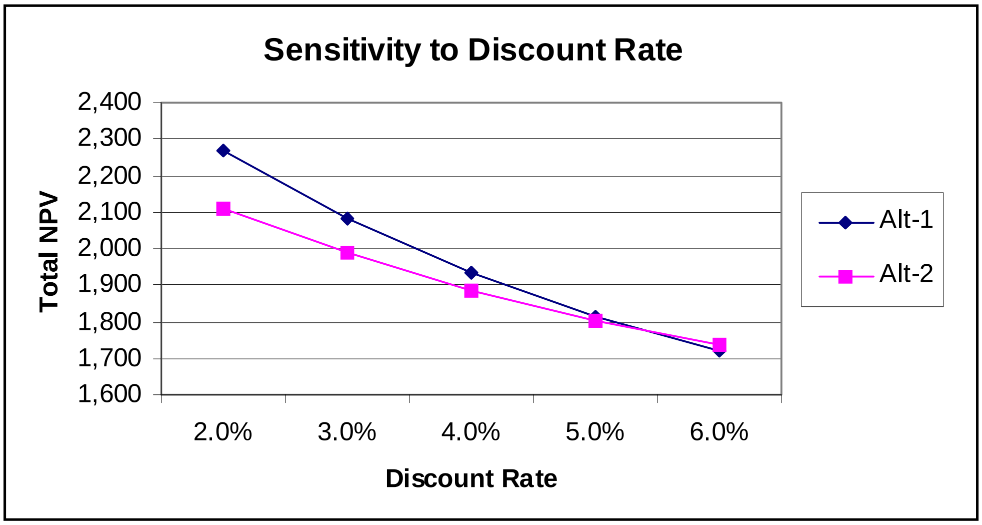


Figure 2.6. Sensitivity of NPV to discount rate.

Table 2.21 separates agency and user cost differences for the same range of discount rates. Inspection of the table 2.21 reveals that Alternative #2 has a higher agency cost than Alternative #1 at all discount rates considered. Further, Alternative #2 has lower user cost than Alternative #1 at all discount rates considered.

Table 2.21. Sensitivity to user cost and discount rate.

Cost Component	Discount Rate, %				
	2.0	3.0	4.0	5.0	6.0
Alternative #1 Agency Cost	1,334	1,281	1,238	1,201	1,171
Alternative #2 Agency Cost	1,413	1,366	1,326	1,292	1,264
Agency Cost Advantage Alt #2 vs #1	-79	-85	-88	-91	-93
Alternative #1 User Cost	932	800	696	613	547
Alternative #2 User Cost	699	621	561	513	475
User Cost Advantage Alt #2 vs #1	233	179	135	100	72
Incremental Benefit/Cost (#2 vs #1)	2.95	2.11	1.53	1.10	0.77

The decision to include or exclude user costs can significantly affect the LCCA results. In an effort to put the agency and user costs in perspective, the bottom row of table 2.21 includes an incremental B/C comparison of the reduction in user costs as a function of increased agency costs. The incremental B/C data in table 2.21 is computed by dividing the reduction in user costs (i.e., benefits) associated with selecting Alternative #2 in lieu of Alternative #1 by the added agency cost(s) associated with selection of Alternative #2.

Similar sensitivity analyses could be conducted using other input variables such as agency cost, user costs, pavement performance lives, and hourly dollar value of user delay.

In addition to conducting a sensitivity analysis, the analyst should examine the implications of contractor work hours on queuing costs as well as the anticipated maximum queue lengths and delay times. Chapter 3 details queue lengths and discusses associated user costs.

A primary drawback of the sensitivity analysis is that the analysis gives equal weight to any input value assumptions, regardless of the likelihood of occurring. In other words, the extreme values (best case and worst case) are given the same likelihood of occurrence as the expected value—which is not realistic. Chapter 4 presents a powerful analytical technique to overcome this limitation.

REEVALUATE DESIGN STRATEGY

Once the net present values have been computed for each alternative and limited sensitivity analysis performed, the analyst needs to step back and reevaluate the competing design strategies. As noted in chapter 1, the overall benefit of conducting a life-cycle cost analysis is not necessarily the LCCA results themselves, but rather how the designer can use the information resulting from the analysis to modify the proposed alternatives and develop more cost-effective strategies.

For example, if user costs dwarf agency costs for all the alternatives, the analysis may indicate that none of the alternatives analyzed are viable. It could indicate that the designer needs to evaluate the current design strategies' impacts on future maintenance of traffic . . . that the design strategies should reflect the need for additional capacity in the out-years to mitigate the impact on highway users. The solution to out-year capacity problems could include enhanced structural design of the shoulders in early-year pavement designs to allow for the use of the shoulder in subsequent rehabilitation traffic control plans. It also could include enhanced structural design of the mainline pavement to minimize the frequency of subsequent rehabilitation efforts and *designing in* features that will make future rehabilitation proceed more smoothly. Other options available include revising the maintenance of traffic plans, reducing the construction period, restricting the contractor's work hours or imposing lane rental fees, planning for additional lanes/routes, and even examining programs to temporarily shift traffic to alternative modes of travel. It is important to note that restricting the contractor's hours of operation or the number of work days allowed will more than likely increase agency cost.

LCCA results are just one of many factors that influence the ultimate selection of a pavement design strategy. The final decision may include a number of additional factors outside the LCCA process, such as local politics, availability of funding, industry capability to perform the required construction, and agency experience with a particular pavement type, as well as the accuracy of the pavement design and rehabilitation models. Chapter 3 of the 1993 *AASHTO Pavement Design Guide* further discusses such other factors.(13) When such *other factors* weigh heavily in the final pavement design selection, it is imperative to document their influence on the final decision.

Many assumptions, estimates, and projections feed the LCCA process. The variability associated with these inputs can have a major influence on the confidence the analyst can place in LCCA results. It all depends on the accuracy of the inputs used. The accuracy of LCCA results depends directly on the analyst's ability to accurately forecast such variables as future costs, pavement performance, and traffic for more than 30 years into the future. To effectively deal with the uncertainty associated with such forecasts, a probabilistic risk analysis approach (as presented in chapter 4) is increasingly essential to quantitatively capture the uncertainty associated with input parameters in LCCA results.

CHAPTER 3. WORK ZONE USER COSTS

This chapter presents a rational, step-by-step procedure to allow the analyst, based on capacity flow analysis, to determine user costs associated with establishing a work zone. Chapter 2 discussed user cost *rates*. This chapter focuses on calculating the quantity associated with each of the individual user cost components (VOC, user delay, and crashes) and computing overall user costs in a spreadsheet environment. The chapter ends with a discussion of automated computer programs.

While there are microcomputer-based user cost analysis software programs currently available, it is important that the analyst thoroughly understand the principles involved before attempting to apply them. By understanding the major factors influencing work zone user costs, the analyst can take steps to minimize the effect of planned future rehabilitation activities on highway users.

WORK ZONE USER COSTS

Work zone user costs are the increased VOC, delay, and crash costs to highway users resulting from construction, maintenance, or rehabilitation work zones. They are a function of the timing, duration, frequency, scope, and characteristics of the work zone; the volume and operating characteristics of the traffic affected; and the dollar cost rates assigned to vehicle operating, delay, and crashes. The following sections address each of these issues.

In the end, user costs are computed by multiplying the quantity of additional VOC, delay, and number of crashes by the unit cost rates assigned to these components.

WORK ZONE DEFINED

Work Zone is defined in the *Highway Capacity Manual (HCM)* as an area of a highway where maintenance and construction operations impinge on the number of lanes available to traffic or affect the operational characteristics of traffic flowing through the area.(14)

Each work zone established over the analysis period affects traffic flow and associated user costs differently and must be evaluated as a separate event. A separate work zone must be defined and analyzed whenever characteristics of the work zone or the characteristics of the affected traffic change.

Pavement design performance differences directly affect the frequency and timing of maintenance and rehabilitation activities. Pavement rehabilitation and maintenance activities generally occur at different points in the analysis period with different traffic, and they generally vary in scope and duration. The time that they occur also affects the influence of the discount factor used in developing NPV.

WORK ZONE CHARACTERISTICS

In order to analyze work zone user costs, work zone characteristics associated with alternative designs and supporting maintenance and rehabilitation strategies must be defined as part of the development of alternative pavement design strategies. Alternative pavement preservation strategies must include how often (the number of times) the facility will be under construction, maintenance or rehabilitation activities, and the year at which work zones are anticipated. Strategies should also include estimates of the number of days the work zone will last, the hours of the day the work zone will be in place, and the anticipated maintenance of traffic strategy.

Work zone characteristics of concern include such factors as work zone length, number and capacity of lanes open, duration of lane closures, timing (hours of the day, days of the week, season of the year, etc.) of lane closures, posted speed, and the availability and physical and traffic characteristics of alternative routes. The strategy for maintaining traffic should include any anticipated restrictions on contractor's or maintenance force's hours of operations or ability to establish lane closures. Specific details in an LCCA should include:

- Projected year work zones occur (years 5, 8, 12, etc,).
- Number of days the work zone will be in place (construction period).
- Specific hours of each day, as well as the days of the week the work zone will be in place.
- Work zone length and posted speed.

The duration of a work zone (the overall length of time a facility or portion of a facility is out of service or traffic is restricted) can range from sporadic daily lane closures for maintenance to several months for bridge deck replacements.

As noted earlier, the differential routine maintenance cost between alternative pavement design strategies tends to be insignificant when compared to initial construction and rehabilitation costs. To a great extent, the same is true of user costs resulting from routine reactive-type maintenance activities. Routine maintenance work zones tend to be relatively infrequent, of short duration, and outside of peak traffic flow periods. As such, analysts should focus attention on user costs associated with major work zones.

TRAFFIC CHARACTERISTICS

User costs are directly dependent on the volume and operating characteristics of the traffic on the facility. Each construction, maintenance, and rehabilitation activity generally involves some temporary effect on traffic using the facility. The effect can vary from insignificant for minor work zone restrictions on low-volume facilities to highly significant for major lane closures on high-volume facilities.

The major traffic characteristics of interest for each year a work zone will be established include: (1) the overall projected Average Annual Daily Traffic (AADT) volumes on both the facility and possibly alternate routes, (2) the associated 24-hour directional hourly demand distributions, and (3) the vehicle classification distribution of the projected traffic streams. On high-volume routes, distinctions between weekday and weekend traffic demand and hourly distributions become important. Further, seasonal AADT traffic distribution also becomes important when work zones are proposed on recreational routes during seasonal peak periods.

AADT

Current AADT volumes are normally readily available for the base year and projected compound traffic growth rates can be obtained from the SHA traffic monitoring section. From these two pieces of information, calculating future year work zone AADT is relatively simple.

While the calculations for future year AADT are rather straightforward, there are two major issues associated with the reasonableness of such projections on high-volume urban facilities. The first is whether the roadway in question can handle large projected volume increases; the second is whether traffic using a facility will continue to use it when work zones are established and traffic flow is restricted.

In the first issue, design capacity and ultimate capacity are quite different. Design capacity is generally set to handle the design year 30 highest hour volume at level of service D or E. However, table 2.1 of the HCM shows measured maximum 24-hour traffic volumes for some of the more heavily traveled interstate urban freeways that far exceed the traffic volumes normally associated with level of service D or E. Reasonableness dictates that AADT projections should not exceed the maximum observed 24-hour traffic volumes contained in table 2.1 of the HCM.

The next section discusses the second issue, which relates to whether traffic using a facility will continue to use it when work zones are established and traffic flow is restricted.

Traffic Diversion

Traffic demand is generally determined based on facility operating characteristics during periods of normal operations. Traffic demand during work zone operations may or may not be the same. Some portion of the traffic normally wanting to use the facility may divert to other routes when work zones are established.

Vehicles use a given facility because it offers what the vehicle operators perceive as the least expensive combination of vehicle operating and time delay costs, consistent with safety requirements. When faced with restricted flow, or even the anticipation of restricted flow, vehicle operators who normally use a facility will exercise one of several options, which are somewhat whimsically categorized below.

Hang Toughers will continue to use the facility as they always have. They are primarily users with little or no option. They (1) must make the trip; (2) must make it at a specific time; and (3) either don't know or don't have alternative routes or modes from which to choose. These users pay the full price of the work zone and have little effect on other facilities in the corridor. In rural areas the predominate choice of through traffic will be to tough it out, as these users generally must make the trip and do not have much information on alternative routes unless formal detours are established.

Time Shifters have the ability and choose to travel on the facility at a different time — generally a time well outside of the restricted flow period. These users lessen their impact by sharing the impact with other vehicles by *invading* their time slot. These users also have little effect on other highway facilities in the corridor, but they do affect hourly traffic distribution.

Detourees either seek out and use alternate routes or are forced to negotiate detours established by the highway agency. These operators also lessen their impact by sharing the impact with other vehicles by *invading* their routes. They tend to trade off anticipated time delay for additional travel distances and associated vehicle operating costs. In urban areas this could include users who switch modes. Detourees can have significant impact on overall user costs of alternative routes, especially those operating at or near capacity. The sections on *Circuitry* (page 69) and *Crash Costs* (page 70) address the additional user costs associated with the additional mileage traveled by detoured and diverted traffic.

Trip Swappers have the luxury of totally abandoning the trip or seeking other destinations when the cost, in terms of time and money, becomes too great. Historically, this group consists primarily of shopping and social/recreational trip makers. While their behavior may diminish the user cost impact of the work zone, they adversely affect businesses along the route in question. More recent trends in people working out of the home and telecommuting may significantly affect work trips in the future.

Handling Diversion

In simple cases, where either work zone disruption is tolerable or alternative routes are limited, estimated AADT during the year of the work zone can be anticipated to continue on the facility and the work zone analysis can be limited to the existing facility.

In more complex situations where existing traffic would face intolerable work zone disruptions, it is entirely possible that total travel demand and hourly distribution on the facility may change when the work zone is established. When demand changes, the scope of work zone user cost analysis may have to expand beyond the existing facility and include user cost changes on major alternative routes. When preliminary analysis of travel demand shows that work zone user delays are unreasonably high, the analyst should seek special help from the traffic engineering section to generate revised traffic demand forecasts during work zone operations.

An alternative approach is to assume that those extraordinary user costs are truly there, and time and route shifters are merely diminishing their effect by sharing the costs with users of other routes and time slots. In short, the misery's still there — it's just being shared differently.

Vehicle Classification

Highway user costs are a composite of the costs of all affected highway users. Highway users are not a homogeneous group. They include commercial and noncommercial vehicles ranging from passenger vehicles through the heaviest trucks. These different vehicle types have different operating characteristics and associated operating costs. Further, the value of user delay differs between vehicle classes. As a result, user costs need to be analyzed for each major vehicle class present in the traffic stream.

There are many truck-vehicle classifications representing various size and weight configurations. Appendix A of the *FHWA Traffic Monitoring Guide*, includes 13 different vehicle classifications.(15) User cost analysis based on 13 vehicle classes is much too detailed for the level of sophistication in the analysis procedures presented.

A more reasonable approach is to use three broad vehicle classes:

1. Passenger cars and other 2-axle, 4-tired passenger vehicles (classification types 1-3)
2. Single-unit trucks, 2-axle, 4-tired or more commercial trucks (classification types 4-7)
3. Combination-unit trucks (classification types 8-13)

On high traffic volume facilities, user delay costs are likely to represent a very significant component of overall user costs in a pavement design LCCA, particularly when vehicle demand exceeds capacity. As passenger vehicles represent the bulk of vehicles in the traffic stream, analysts may find it beneficial to subdivide passenger vehicles into commercial and noncommercial categories. Further, although vehicle occupancy rates have consistently fallen over time, they cannot generally be ignored.

Directional Hourly Traffic Distribution

The estimated hourly traffic distribution during work zone operations is essential to compare the unrestricted demand on the facility with the facility's ability to carry that traffic through the work zone. Hourly distribution specific to a particular facility can be developed from agency traffic data or general distribution data developed for various functional class- and area-type combinations. On high-volume or tourist destination routes, the difference between weekday and weekend traffic volumes and hourly distributions needs to be considered.

Table 3.1 represents default directional hourly distributions for *Urban* and *Rural* roadways included in MicroBENCOST. The table also includes hourly directional distribution factors associated with inbound and outbound directions. The hourly demand is computed by multiplying the AADT by the hourly percent and directional factors for the direction in question.

Hourly Demand = AADT x Hourly Distribution Factor x Hourly Directional Factor

Table 3. 1. Default hourly distributions from MicroBENCOST (all functional classes).

Hour (24-Hr Clock)	Rural			Urban		
	% ADT	Direction %		% ADT	Direction %	
		In	Out		In	Out
0 – 1	1.8	48	52	1.2	47	53
1 – 2	1.5	48	52	0.8	43	57
2 – 3	1.3	45	55	0.7	46	54
3 – 4	1.3	53	47	0.5	48	52
4 – 5	1.5	53	47	0.7	57	43
5 – 6	1.8	53	47	1.7	58	42
6 – 7	2.5	57	43	5.1	63	37
7 – 8	3.5	56	44	7.8	60	40
8 – 9	4.2	56	44	6.3	59	41
9 – 10	5.0	54	46	5.2	55	45
10 – 11	5.4	51	49	4.7	46	54
11 – 12	5.6	51	49	5.3	49	51
12 – 13	5.7	50	50	5.6	50	50
13 – 14	6.4	52	48	5.7	50	50
14 – 15	6.8	51	49	5.9	49	51
15 – 16	7.3	53	47	6.5	46	54
16 – 17	9.3	49	51	7.9	45	55
17 – 18	7.0	43	57	8.5	40	60
18 – 19	5.5	47	53	5.9	46	54
19 – 20	4.7	47	53	3.9	48	52
20 – 21	3.8	46	54	3.3	47	53
21 – 22	3.2	48	52	2.8	47	53
22 – 23	2.6	48	52	2.3	48	52
23 – 24	2.3	47	53	1.7	45	55

For example, the inbound directional hourly demand volume between 8 and 9 a.m. on an urban roadway with a 40,000 AADT would be:

40,000 AADT x 6.3% x 59% = 1,487 vehicles/hour

Although MicroBENCOST provides for the use of different directional hourly distribution values for *Interstate, Principal Arterial, Minor Arterial, and Major Collector*, the default values listed are the same for all functional classes. Pennsylvania, on the other hand, provides different hourly distributions for various functional class roadways, but it does not provide a directional split factor. Table 3.2 contains statewide hourly distribution factors used by PennDOT in developing user delay cost.

Table 3.2. PennDOT AADT distribution (hourly percentages).

Hour (24 Hr Clock)	Traffic Pattern Group					
	Interstate		Principal Arterial		Minor Arterial	
	Urban	Rural	Urban	Rural	Urban	Rural
12 – 1	1.3	1.7	0.9	0.9	0.8	0.7
1 – 2	0.9	1.4	0.5	0.5	0.4	0.4
2 – 3	0.8	1.3	0.4	0.5	0.3	0.3
3 – 4	0.8	1.3	0.4	0.5	0.3	0.4
4 – 5	1.1	1.4	0.6	0.9	0.4	0.8
5 – 6	2.1	2.1	1.8	2.3	1.3	2.2
6 – 7	4.7	3.7	4.4	4.9	4.0	4.5
7 – 8	6.4	4.9	6.2	6.2	6.4	5.5
8 – 9	5.6	4.9	5.7	5.5	5.7	5.3
9 – 10	5.1	5.2	5.1	5.3	4.8	5.4
10 – 11	5.2	5.5	5.2	5.4	4.9	5.8
11 – 12	5.4	5.8	5.6	5.6	5.5	6.0
12 – 13	5.5	5.7	6.0	5.7	6.0	6.2
13 – 14	5.5	5.9	5.9	5.9	5.7	6.4
14 – 15	6.1	6.3	6.4	6.6	6.3	7.2
15 – 16	7.3	6.9	7.4	7.7	7.6	8.1
16 – 17	7.8	7.2	7.8	8.0	8.3	8.0
17 – 18	7.2	6.6	7.5	7.4	8.0	7.1
18 – 19	5.4	5.3	5.9	5.5	6.2	5.4
19 – 20	4.3	4.4	4.8	4.3	5.1	4.4
20 – 21	3.7	3.8	4.0	3.6	4.3	3.6
21 – 22	3.2	3.4	3.3	3.0	3.4	2.9
22 – 23	2.6	2.9	2.4	2.3	2.4	2.1
23 – 24	2.0	2.4	1.7	1.5	1.6	1.4

CONCEPTUAL ANALYSIS

Before addressing user cost calculation procedures, it is helpful to conduct a conceptual analysis of a work zone operation. There are seven possible work zone user cost components that can occur; three are associated with a *Base Case* situation where traffic operates under *Free-Flow* conditions, and four are associated with a *Queue* situation where traffic operates under *Forced-Flow* conditions. The next section conceptually discusses potential user costs involved under Free-Flow and Forced-Flow (Level of Service F) conditions.

Free Flow

Work zones restrict traffic flow either by reducing capacity or, as a minimum, by posting lower speed limits. Figure 3.1 shows free-flow conditions at a work zone. All traffic that flows through the work zone must, at a minimum, slow while traveling through it and then accelerate back to normal operating speed. This is commonly referred to as a speed change and it results in three work zone-related User Cost components: speed change delay, speed change VOC, and reduced speed delay. Figure 3.1 presents the cost components associated with free flow.

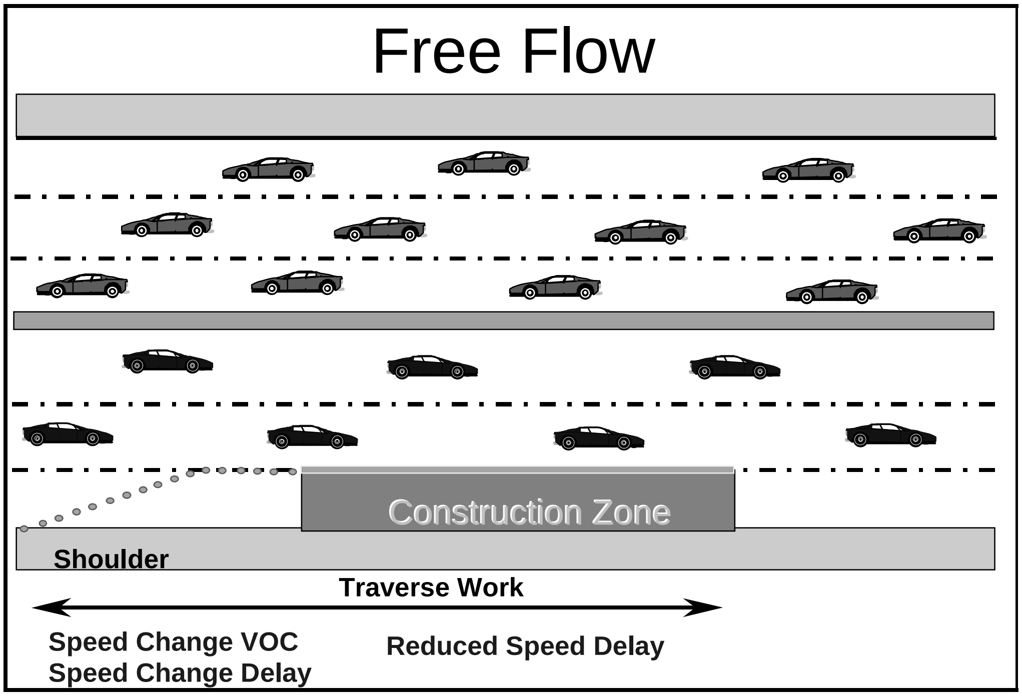


Figure 3.1. Free-flow cost components.

Speed Change Delay is the additional time necessary to decelerate from the upstream approach speed to the work zone speed and then to accelerate back to the initial approach speed after traversing the work zone.

Speed Change VOC is the additional vehicle operating cost associated with decelerating from the upstream approach speed to the work zone speed and then accelerating back to the approach speed after leaving the work zone.

Reduced Speed Delay is the additional time necessary to traverse the work zone at the lower posted speed; it depends on the upstream and work zone speed differential and length of the work zone.

If traffic demand remains below work zone capacity, added work zone user costs are limited to the above three components and the analysis is relatively simple. In most cases, delay times remain relatively low and represent more of a minor irritation and inconvenience than a serious problem.

Forced Flow (Level of Service F)

When hourly traffic demand exceeds work zone capacity, traffic flow breaks down and a queue of vehicles develops, as figure 3.2 shows. It is important to note that the queue forms not in the work zone itself, but in the upstream approach to the work zone.

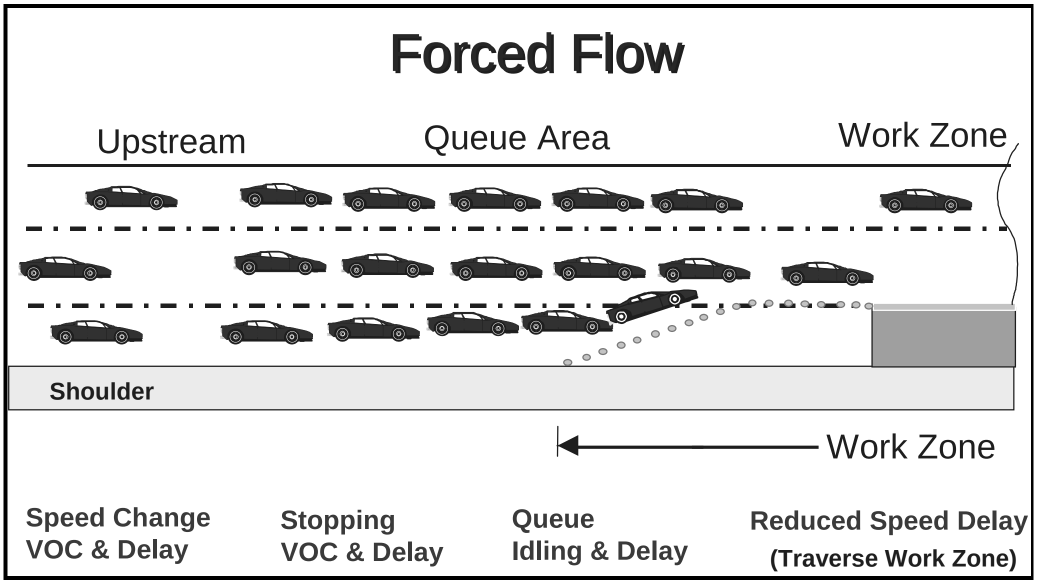


Figure 3.2. Forced-flow cost components level of service F.

Once a queue develops, all approaching vehicles must not only slow down before proceeding through the work zone itself, but they also must stop at the upstream end of the queue and creep through the length of the physical queue under forced-flow conditions. As long as demand exceeds capacity, the length of the queue grows, exacerbating the problem. When demand eventually falls below capacity, or when capacity is increased above demand by removing the work zone restriction, vehicles then leave the queue faster than they arrive and the length of the queue shrinks and eventually dissipates. When capacity is reduced on high-traffic facilities, it is common for queues to develop in the morning peak traffic period, dissipate, and then redevelop in the afternoon peak traffic period. In exceptionally congested areas, queues may form early in the morning and continue throughout the day and into the evening hours.

Queuing situations impose four work zone-related user costs that only apply to vehicles that encounter a physical queue.

Stopping Delay is the additional time necessary to come to a complete stop from the upstream approach speed (instead of just slowing to the work zone speed) and the additional time to accelerate back to the approach speed after traversing the work zone.

Stopping VOC is the additional vehicle operating cost associated with stopping from the upstream approach speed and accelerating back up to the approach speed after traversing work zone.

Queue Delay is the additional time necessary to creep through the queue under forced-flow conditions.

Idling VOC is the additional vehicle operating cost associated with *stop-and-go* driving in the queue. The idling cost rate multiplied by the additional time spent in the queue is an approximation of actual VOC associated with stop-and-go conditions. When a queue exists, stopping delay and VOC replace the free-flow speed change delay and VOC.

The conceptual analysis presented here is geared primarily to freeway conditions. Conceptual analysis of other facilities with at-grade intersections would also incur speed change, stopping, delay, and idling cost but at a much higher frequency, because of intersection-control devices and turning movements.

COMPUTATIONAL ANALYSIS

Once the individual work zones have been identified, each is evaluated separately. This is the point at which individual user cost components are quantified and converted to dollar cost values. This section provides an approach for actually quantifying and costing the individual work zone user cost components encountered. The 12 overall steps involved are:

1. Project future year traffic demand.
2. Calculate work zone directional hourly demand.
3. Determine roadway capacity.
4. Identify the user cost components.
5. Quantify traffic affected by each component.
6. Compute reduced speed delay.
7. Select and assign VOC cost rates.
8. Select and assign delay cost rates.
9. Assign traffic to vehicle classes.
10. Compute individual user costs components by vehicle class.
11. Sum total work zone user costs.
12. Address circuitry and crash costs.

Example Work Zone Problem Defined

The following information describes the work zone used as the example problem throughout the remainder of this section to illustrate the work zone user cost computational steps described above.

Facility:	3-lane directional outbound Interstate. Posted speed is 55 mi/h and grades on the facility are less than 2 percent.
Traffic:	Base year (1995) AADT of 122,000 vehicles per day. Vehicle classification counts indicate a traffic stream mix of 90 percent Passenger

Vehicles, 5.4 percent Single-Unit Trucks, and 4.6 percent Combination Trucks. The 10-year projected traffic growth rates for the various vehicle classifications are 2.2 percent for passenger vehicles and 5 percent per year for both single-unit and combination trucks.

Work Zone: 5.25-mile-long single-lane closure from 9 a.m. until 3 p.m. and from 8 p.m. until 5 a.m. the following morning. Work zone posted speed is 40 mi/h. The work zone will be in place for 60 days in 1999.

Step 1. Project Future Year Traffic Demand

The first step is to project future year hourly traffic demand volumes for each vehicle class for the year the work zones will be in place, from current or base year AADT, using compound traffic growth factors. The following formula applies:

$$\text{Future Year AADT} = \text{Base Year AADT} \times \text{Vehicle class \%} \times (1 + \text{growth rate})^{(\text{Future Yr.} - \text{Base Yr.})}$$

The AADT on a facility in 1999 can be determined from a 1995 base year AADT of 122,000 using the above formula by applying the appropriate compound growth rate factor (in this example 2.2 percent for passenger vehicles and 5 percent for trucks) as follows:

Projected 1999 AADT

Passenger Vehicles	122,000 x 0.900 x $1.022^{(4)}$	=	109,800 x 1.09095	=	119,786	=	89%
Single-Unit Trucks	122,000 x 0.054 x $1.050^{(4)}$	=	6,588 x 1.21551	=	8,008	=	6%
Combination Trucks	122,000 x 0.046 x $1.050^{(4)}$	=	5,612 x 1.21551	=	6,821	=	5%
Total Traffic					134,615	=	100%

Based on these new numbers, total traffic in 1999 will be 134,615, and because of the differential traffic growth rates for trucks, the 1999 vehicle mix will be approximately 89 percent for passenger vehicles, 6 percent for single-unit trucks, and 5 percent for combination trucks.

Step 2. Calculate Work Zone Directional Hourly Demand

Directional hourly traffic distribution should be determined from agency traffic data on the roadway being analyzed or from traffic data on similar facilities. If, however, such data is not available, the default hourly distributions for various roadway types in urban and rural settings from MicroBENCOST (reproduced in table 3.1) can be used. Table 3.3 contains the future year directional hourly demand for the example problem generated using the default MicroBENCOST urban outbound hourly distribution factors and 134,615 future year AADT.

Inspection of table 3.3 reveals that a.m. outbound demand peaks at 4,200 vehicles per hour in the 7 to 8 a.m. period, while the p.m. outbound demand peaks at 6,885 vehicles per hour in the 5 to 6 p.m. time period.

Table 3.3. Work zone directional hourly demand (all vehicle classes).

AADT 134,615	**Outbound Urban Interstate**		
(24-Hr Clock)	**% ADT**	**Directional Factor %**	**Demand**
12 – 1	1.2	53	856
1 – 2	0.8	57	614
2 – 3	0.7	54	509
3 – 4	0.5	52	350
4 – 5	0.7	43	405
5 – 6	1.7	42	961
6 – 7	5.1	37	2,540
7 – 8	7.8	40	4,200
8 – 9	6.3	41	3,477
9 –10	5.2	45	3,150
10 –11	4.7	54	3,417
11 –12	5.3	51	3,639
12 –13	5.6	50	3,769
13 –14	5.7	50	3,837
14 –15	5.9	51	4,051
15 –16	6.5	54	4,725
16 –17	7.9	55	5,849
17 –18	8.5	60	6,865
18 –19	5.9	54	4,289
19 –20	3.9	52	2,730
20 –21	3.3	53	2,354
21 –22	2.8	53	1,998
22 –23	2.3	52	1,610
23–24	1.7	55	1,259
Total			67,453

Step 3. Determine Roadway Capacity

In analyzing work zone user costs, there are three capacities that need to be determined: (1) the free-flow capacity of the facility under normal operating condition, (2) the capacity of the facility when the work zone is in place, and (3) the capacity of the facility to dissipate traffic from a standing queue.

Free-Flow Capacity is the maximum capacity a facility can handle under free-flow conditions. According to the 1994 HCM (p. 2-10), the maximum capacity for a 2-lane directional freeway under ideal conditions is 2,200 passenger cars per hour per lane (pcphpl) and 2,300 pcphpl for a 3- or more lane directional freeway. The 1994 HCM points out the need to reduce the above ideal condition capacities for such real world factors as restricted lane widths, reduced lateral clearances, the presence of trucks and recreational vehicles, and the presence of a driver population unfamiliar with the area (p. 3-11). The real-world free-flow capacity of the facility is determined by applying the following formula:

$$Sf_i = MSF_i \text{ x } N \text{ x } f_w \text{ x } f_{HV} \text{ x } f_p$$

SFi = service flow rate for LOS i under prevailing roadway and traffic conditions for N lanes in one direction in vehicles per hour (vph),
MSFi = Maximum service flow rate for LOS i for N lanes in one direction (vph)
N = number of lanes in one direction of the freeway,
f_w = factor to adjust for the effects of restricted lane widths and lateral clearances,
f_{HV} = factor to adjust for the effect of heavy vehicles on the traffic stream, and
f_p = factor to adjust for the effect of recreational or unfamiliar driver populations.

The following discussion describes the procedures to adjust maximum freeway capacity in terms of pcplph to real-world mixed vehicle conditions when there are trucks in the traffic stream (the most commonly encountered condition). For adjustments resulting from restricted lane width, reduced lateral clearance, and other adjustment factors when the work zone is not in place, refer to chapter 3 (p. 3-12) of the 1994 HCM.

The capacity adjustment resulting from the presence of trucks is based on the fact that trucks are larger than passenger vehicles and thus physically occupy more roadway space. Further, trucks, particularly when fully loaded, tend to be less nimble and maneuverable than passenger vehicles, especially on long, steep up grades.

Truck equivalency factors are used to adjust highway capacity for the presence of trucks in the traffic stream. Truck equivalency factors are a function of the percent trucks in the traffic stream and the degree and length of the maximum vertical grade on the facility. Table 3.4 reproduces the truck equivalency factors for various combinations of percent trucks, grades, and grade lengths found in table 3-4 of the 1994 HCM.

Inspection of table 3.4 clearly shows that truck equivalency factors increase as grades and grade length increase. The effect of grade and grade length diminishes as the percent of trucks in the traffic stream increases. It is also clear from table 3.4 that at grades of less than 2 percent, the truck equivalency factor is 1.5, regardless of the length of the grade or the percent of trucks in the traffic stream. Because the example problem used throughout this chapter has by definition less than 2 percent grades, the truck equivalency factor for the example problem will be 1.5.

Table 3.4. Truck equivalency factors.

Grade %	Length (Miles)	Percent Trucks and Buses								
		2	**4**	**5**	**6**	**8**	**10**	**15**	**20**	**25**
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
2	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.50 - 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.75 - 1.00	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	1.00 - 1.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 1.50	4.5	3.5	3.0	3.0	2.5	2.5	2.0	2.0	2.0
3	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	3.0	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5
	0.50 - 0.75	6.0	4.0	4.0	3.5	3.5	3.0	2.5	2.5	2.0
	0.75 - 1.00	7.5	5.5	5.0	4.5	4.0	4.0	3.5	3.0	3.0
	1.00 - 1.50	8.0	6.0	5.5	5.0	4.5	4.0	4.0	3.5	3.0
	> 1.50	8.5	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
4	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	5.5	4.0	4.0	3.5	3.0	3.0	3.0	2.5	2.5
	0.50 - 0.75	9.5	7.0	6.5	6.0	5.5	5.0	4.5	4.0	3.5
	0.75 - 1.00	10.5	8.0	7.0	6.5	6.0	5.5	5.0	4.5	4.0
	>1.00	11.0	8.0	7.5	7.0	6.0	6.0	5.0	5.0	4.5
5	0.00 - 0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.33	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0
	0.33 - 0.50	9.0	7.0	6.0	6.0	5.5	5.0	4.5	4.0	3.5
	0.50 - 0.75	12.5	9.0	8.5	8.0	7.0	7.0	6.0	6.0	5.0
	0.75 - 1.00	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
	> 1.00	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
6	0.00 - 0.25	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
	0.25 - 0.33	9.0	6.5	6.0	6.0	5.0	5.0	4.0	3.5	3.0
	0.33 - 0.50	12.5	9.5	8.5	8.0	7.0	6.5	6.0	6.0	5.5
	0.50 - 0.75	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5
	0.75 - 1.00	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5
	> 1.00	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5

Tables 3.5 and 3.6 are maximum freeway capacity (mixed vehicles per lane per hour) *look up* tables based on truck equivalency factor and percent trucks in the traffic stream. Table 3.5 is for 2-lane directional freeways and table 3.6 is for 3- or more lane directional freeways. Table 3.6 is the appropriate table for the example problem. Table 3.6 (6- or more lane facility), with a truck equivalency factor of 1.5 and future year percent trucks of 11 percent, reveals by extrapolation a free-flow capacity of 2,180 vehicles per lane per hour, or 6,540 vph for all 3 lanes.

Table 3.5. Maximum mixed vehicle traffic capacities for trucks in the traffic stream (4-lane facilities).

%	Truck Equivalency Factor									
Trucks	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**	**5.5**	**6.0**
0.0%	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200
2.0%	2,178	2,157	2,136	2,115	2,095	2,075	2,056	2,037	2,018	2,000
4.0%	2,157	2,115	2,075	2,037	2,000	1,964	1,930	1,897	1,864	1,833
5.0%	2,146	2,095	2,047	2,000	1,956	1,913	1,872	1,833	1,796	1,760
6.0%	2,136	2,075	2,018	1,964	1,913	1,864	1,818	1,774	1,732	1,692
8.0%	2,115	2,037	1,964	1,897	1,833	1,774	1,719	1,667	1,618	1,571
10.0%	2,095	2,000	1,913	1,833	1,760	1,692	1,630	1,571	1,517	1,467
12.0%	2,075	1,964	1,864	1,774	1,692	1,618	1,549	1,486	1,429	1,375
14.0%	2,056	1,930	1,818	1,719	1,630	1,549	1,477	1,410	1,350	1,294
15.0%	2,047	1,913	1,796	1,692	1,600	1,517	1,443	1,375	1,313	1,257
16.0%	2,037	1,897	1,774	1,667	1,571	1,486	1,410	1,341	1,279	1,222
18.0%	2,018	1,864	1,732	1,618	1,517	1,429	1,350	1,279	1,215	1,158
20.0%	2,000	1,833	1,692	1,571	1,467	1,375	1,294	1,222	1,158	1,100
22.0%	1,982	1,803	1,654	1,528	1,419	1,325	1,243	1,170	1,106	1,048
24.0%	1,964	1,774	1,618	1,486	1,375	1,279	1,196	1,122	1,058	1,000
25.0%	1,956	1,760	1,600	1,467	1,354	1,257	1,173	1,100	1,035	978

Table 3.6. Maximum mixed vehicle traffic capacities for trucks in the traffic stream (6 or more lanes).

%	Truck Equivalency Factor									
Trucks	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**	**5.5**	**6.0**
0.0%	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300
2.0%	2,277	2,255	2,233	2,212	2,190	2,170	2,150	2,130	2,110	2,091
4.0%	2,255	2,212	2,170	2,130	2,091	2,054	2,018	1,983	1,949	1,917
5.0%	2,244	2,190	2,140	2,091	2,044	2,000	1,957	1,917	1,878	1,840
6.0%	2,233	2,170	2,110	2,054	2,000	1,949	1,901	1,855	1,811	1,769
8.0%	2,212	2,130	2,054	1,983	1,917	1,855	1,797	1,742	1,691	1,643
10.0%	2,190	2,091	2,000	1,917	1,840	1,769	1,704	1,643	1,586	1,533
12.0%	2,170	2,054	1,949	1,855	1,769	1,691	1,620	1,554	1,494	1,438
14.0%	2,150	2,018	1,901	1,797	1,704	1,620	1,544	1,474	1,411	1,353
15.0%	2,140	2,000	1,878	1,769	1,673	1,586	1,508	1,438	1,373	1,314
16.0%	2,130	1,983	1,855	1,742	1,643	1,554	1,474	1,402	1,337	1,278
18.0%	2,110	1,949	1,811	1,691	1,586	1,494	1,411	1,337	1,271	1,211
20.0%	2,091	1,917	1,769	1,643	1,533	1,438	1,353	1,278	1,211	1,150
22.0%	2,072	1,885	1,729	1,597	1,484	1,386	1,299	1,223	1,156	1,095
24.0%	2,054	1,855	1,691	1,554	1,438	1,337	1,250	1,173	1,106	1,045
25.0%	2,044	1,840	1,673	1,533	1,415	1,314	1,227	1,150	1,082	1,022

Queue Dissipation Rates

Capacity during queue dissipation is less than the capacity for free-flow conditions, even though the lanes are unrestricted (1994 HCM, 2-29). The reduction can easily be as much as 200 vph. According to the 1994 HCM, “. . . various observations of freeway queue departure rates range from as low as 1,500 pcphpl to as high as 2,000 pcphpl.” This implies that a separate and distinct temporary *dissipation capacity* rate exists after a work zone is removed. This rate comes into play when work zones are only in place for certain hours of the day (i.e, when work zones are removed during peak traffic flow periods).

Removal of restrictions in front of a queue can also be analyzed much like the dissipation of a queue at a red traffic signal. That is, the first cars move out rather slowly while follow-on cars take a little less time and finally stabilize at a saturation flow rate further back in the queue. Table 3.7 shows observed saturation flow rates (capacity) departing from traffic signals as given in table 2-13, page 2-32, of the 1994 HCM.

As noted earlier, various observations of freeway queue departure rates range from as low as 1,500 pcphpl to as high as 2,000. Using an average of 1,818, with a standard deviation of 144 from analysis of the traffic signal analogy above, there is a 68 percent probability that the queue dissipation rate would be somewhere between 1,674 and 1,962. Alternately, there is a 95.5 percent probability that it would be somewhere between 1,530 and 2,106. Further discussion of queue departure rates can be found on page 6-7 of the 1994 HCM.

Table 3.7. Observed saturation flow rates per hour of green time.

1,470	1,572	1,651	1,682	1,785	1,791
1,832	1,840	1,875	1,827	1,896	1,905
1,910	1,936	1,937	2,000	2,000	-
Average					1,818
Standard Deviation					144

Using a 50 percent reliability, the queue dissipation capacity selected for the example problem is 1,818 vehicles per lane. With 3 lanes open, total dissipation capacity becomes 5,454 vph.

Work Zone Capacity

Traffic capacity in the work zone can be estimated from research on the capacity associated with various lane closures on multilane facilities. Table 3.8 reflects observed work zone mixed vehicle flow capacities at several real-world work zones under several lane closure scenarios.(16)

Table 3.8. Measured average work zone capacities.

Directional Lanes		Number of Studies	Average Capacity	
Normal Operations	Work Zone Operations		Vehicles Per Hour	Vehicles per Lane per Hour
3	1	7	1,170	1,170
2	1	8	1,340	1,340
5	2	8	2,740	1,370
4	2	4	2,960	1,480
3	2	9	2,980	1,490
4	3	4	4,560	1,520

Table 3.8 indicates, for example, that a 3-lane directional facility with 1 lane closed and 2 lanes opened to traffic (line 5 of table 3.8) will have a total average work zone capacity of 2,980 vph, and only 1,490 vehicles per lane per hour. As the 1,490 vehicles per lane per hour represent the mean capacity, it incorporates a 50 percent reliability factor (i.e., half of the time the capacity will be greater than 1,490 and half the time less than 1,490). Figure 3.3 shows the capacity ranges observed for the average work zone capacities in table 3.8. This same data is plotted as a descending cumulative probability distribution in figure 3.4.

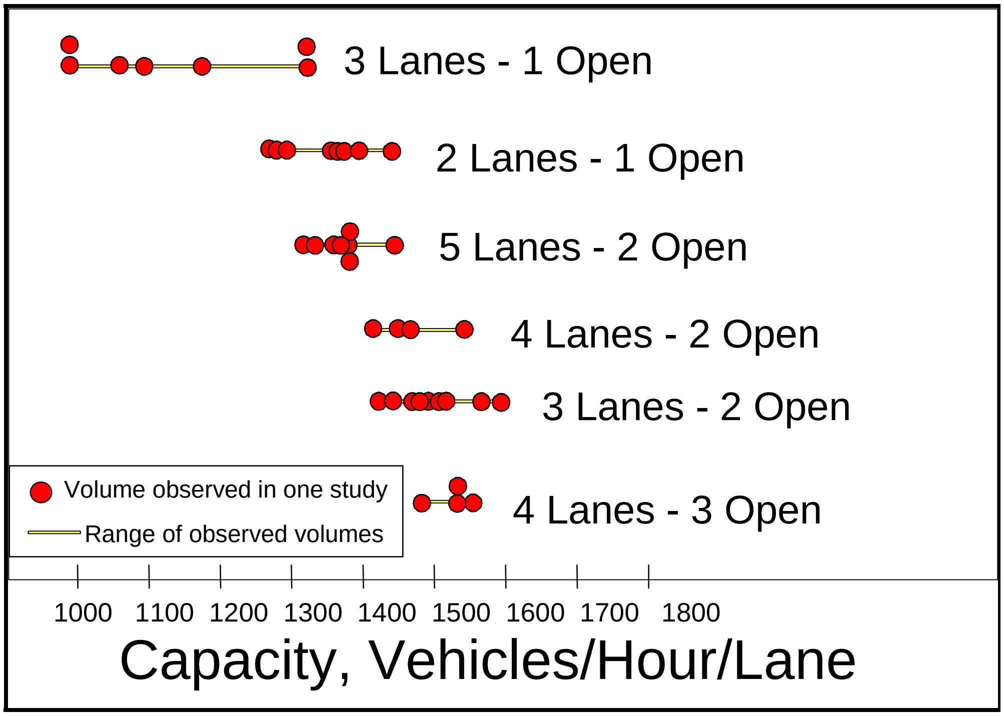


Figure 3.3. Range of observed work zone capacities.

Figure 3.4 is used to incorporate a reliability factor in the value selected for the work zone capacity. Figure 3.4 is used by selecting the desired percent reliability factor from the Y axis, then intersecting the appropriate work zone situation, and estimating the corresponding capacity. The x-axis intercept represents the adjusted work zone directional mixed vehicle flow capacity per lane for the work zone configuration and reliability factor selected.

For the example problem, an 80 percent reliability factor will be used to determine work zone capacity. By entering the figure at an 80 percent reliability and intersecting the curve for a 3-lane directional facility with 2 lanes open, the work zone capacity, determined by inspection, is approximately 1,415 vehicles per lane or 2,830 vph. Using an 80 percent reliability is roughly equivalent to saying that the work zone capacity will be at least equal to 2,830 vehicles per hour 80 percent of the time. It also means, however, that the capacity of the work zone can be less than 2,830 for 20 percent of the time.

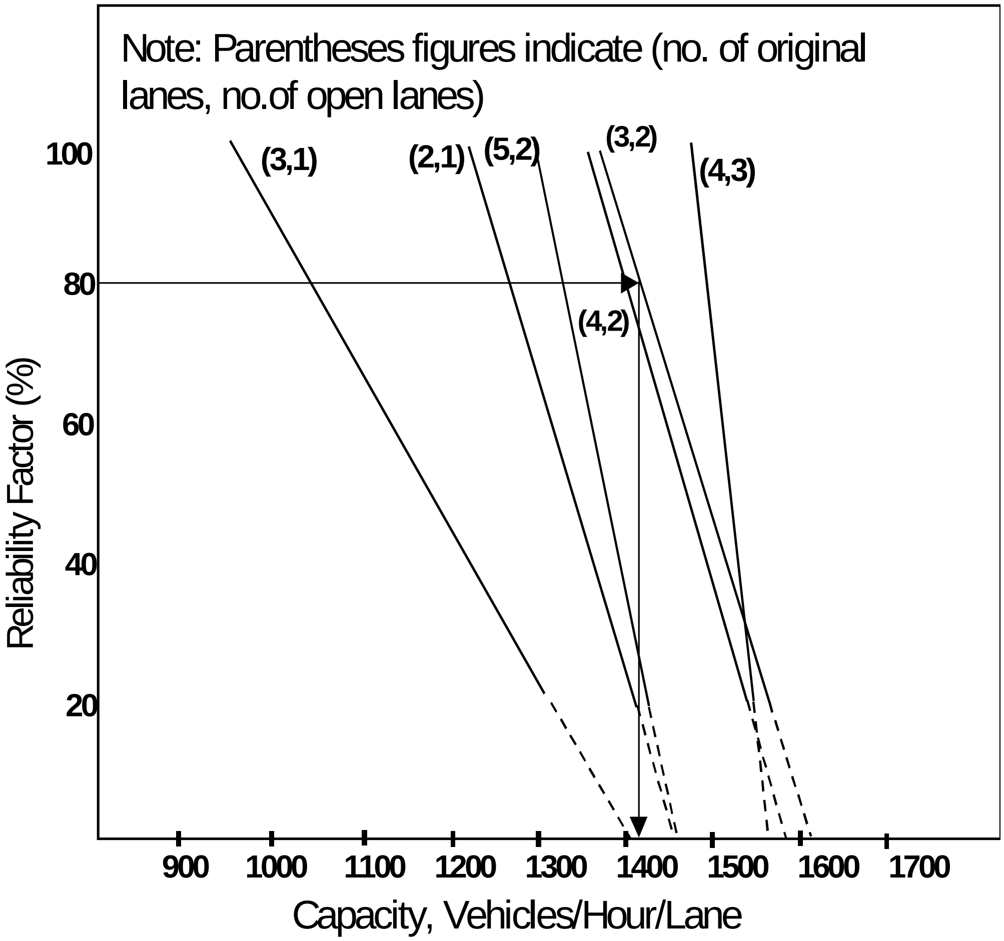


Figure 3.4. Cumulative distribution of observed work zone capacities. (Source: HCM, 1994)

Step 4. Identify the User Cost Components

With the roadway capacities established, the fourth step is to compare the roadway capacity with the hourly demand for the facility. One of the most difficult problems in analyzing work zone user costs is keeping track of all the input values and the resulting computations. One of the most effective methods of keeping track is to set up the problem in a microcomputer spreadsheet software program. Most of the following tables, such as table 3.9, were created in a spreadsheet software program. It provides a convenient way to compare capacity and hourly demand, and it forms the basis for determining the user cost components that come into play.

Table 3.9. Work zone analysis matrix.

AADT 134,615			Queue Rate	Num. of Queued Vehicles	Lanes Open	Operating Conditions	Cost Factors
Hour (24-Hr Clock)	Demand	Capacity					
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)
0 – 1	856	2,830	-1,974	0	2	Free Flow WZ in place No Queue	Free Flow Only Costs (3)
1 – 2	614	2,830	-2,216	0			
2 – 3	509	2,830	-2,321	0			
3 – 4	350	2,830	-2,480	0			
4 – 5	405	2,830	-2,425	0			
5 – 6	961	6,540	-5,579	0	3	Free Flow No WZ No Queue	No Costs
6 – 7	2,540	6,540	-4,000	0			
7 – 8	4,200	6,540	-2,340	0			
8 – 9	3,477	6,540	-3,063	0			
9 –10	3,150	2,830	320	320	2	Forced Flow WZ in place Queue Exists	WZ Delay and Queuing (5 costs)
10 –11	3,417	2,830	587	907			
11 –12	3,639	2,830	809	1,715			
12 –13	3,769	2,830	939	2,654			
13 –14	3,837	2,830	1,007	3,661			
14 –15	4,051	2,830	1,221	4,881			
15 –16	4,725	5,454	-729	4,152	3	Forced Flow No WZ Queue Exists	Queuing Only (4 costs)
16 –17	5,849	5,454	395	4,548			
17 –18	6,865	5,454	1,411	5,959			
18 –19	4,289	5,454	-1,165	4,794			
19 –20	2,730	5,454	-2,724	2,070			
20 –21	2,354	2,830	-476	1,594	2	Forced Flow WZ in place Queue Exists	WZ Delay and Queue (5 Costs)
21 –22	1,998	2,830	-832	762			
22 –23	1,610	2,830	-1,220	0			
23 –24	1,259	2,830	-1,571	0		No Queue	Free Flow
Total	67,453						

Note: Shaded areas represent hours the work zone is in place.

In table 3.9, column (b) shows the directional hourly travel demand determined in table 3.3 on page 43, while column (c) shows the capacities just determined. The capacities shown include:

(1) Work zone capacity of 2,830 vph when the work zone is in place (midnight to 5 a.m., 9 a.m. to 3 p.m., and once again from 8 p.m. to midnight),

(2) Free-flow capacity of 6,540 vph during free flow operation when the work zone is not in place (5 to 9 a.m.), and

(3) Queue dissipation capacity of 5,454 vph for the 3 to 8 p.m. time period when the work zone is removed but a built-up queue exists and starts to dissipate.

Because the major work zone user cost impact is the result of the user delay component of traversing any queue that may develop, it is important to determine whether or not a queue will form, and if it forms, how long it will take to dissipate. To answer this question, the directional hourly demand is compared to the available capacity ***for each hour of the day***.

The *Queue Rate* in column (d) is the difference between hourly capacity of the facility and the unrestricted hourly demand (demand minus capacity) during each hour of the day. The queuing rate is the hourly *rate* at which vehicles accumulate to, or, if negative, dissipate from any queue that may exist. Column (e), on the other hand, represents the *cumulative* number of vehicles *backed up* in the queue at the end of each hour.

To assist in the analysis, table 3.9 includes several additional temporary columns:

(1) Column (f) indicates the number of lanes open to traffic.
(2) Column (g) describes the facilities' operating conditions (free or forced flow).
(3) Column (h) describes the user cost factors that apply during each hour of the day.

The most comprehensive approach in analyzing table 3.9 would be to conduct an hour-by-hour analysis. The analysis can be significantly simplified by grouping and analyzing hours of the day with similar operating characteristics. The analysis below centers around 5 periods of the day (one is repeated) that share similar operating characteristics.

Period 1

Inspection of table 3.9 shows the work zone is in place from midnight until 5 a.m. and that capacity is restricted to 2,830 vph. As traffic demand is lower than capacity, the facility operates under free-flow conditions. There is no queue and no vehicles have to stop. Under these conditions the work zone results in three free-flow user costs: the VOC and delay cost of the speed change associated with slowing down for the work zone, and the delay cost of traversing the work zone at a reduced speed.

Period 2

From 5 to 9 a.m. the work zone is removed and, with no built-up queue, capacity increases to the free-flow rate of 6,540 vph and exceeds demand throughout the period. Under these conditions, there’s no queue and no work zone, and therefore there are no work zone- or queue-related user costs.

Period 3

At 9 a.m., when the work zone is reestablished, capacity falls back to 2,830 vph until 3 p.m., when the work zone is removed for the evening rush hours. During this period demand exceeds capacity and a queue forms. By 10 a.m. the queue grows to 320 vehicles and continues to grow to 4,881 vehicles by 3 p.m., when the work zone is again removed. During the 9 a.m. to 3 p.m. period, there are a total of five user cost components. They are the four *forced-flow* user costs associated with queuing (stopping VOC and delay costs, idling VOC, and delay cost of crawling through the queue) as well as the free-flow delay in traversing the work zone. The speed change delay and VOC cost factors have been replaced by the delay and VOC stopping cost factors.

Period 4

The work zone is removed at 3 p.m. Because there are 4,881 vehicles already queued on the roadway, capacity only increases to 5,454 vph (the queue dissipation rate), rather than the normal free-flow capacity of 6,540 vph.

From 3 to 4 p.m., demand is less than capacity and the queue starts to dissipate, falling to 4,152 vehicles at 4 p.m. Rush hour demand from 4 to 6 p.m. again exceeds capacity. The queue grows again to its maximum length of 5,959 at 6 p.m. before dissipating over the next 2 hours to 2,070 vehicles at 8 p.m., when the work zone is reestablished. During this entire period from 3 until 8 p.m., there is no work zone and therefore no free-flow user cost. There are, however, queuing user costs (stopping VOC and delay costs, and idling and delay cost of crawling through the queue).

Period 5

At 8 p.m., when the work zone is reestablished, there is a standing queue of 2,070 vehicles. While the capacity falls to 2,830 vph, it exceeds the hourly demand and the queue continues to dissipate. The queue finally completely dissipates somewhere around 10:30 p.m. During this 8 to 11 p.m. period, conditions are identical to period 3.

Period 1 … Revisited

Finally, by 11 p.m. the queue is completely gone and traffic flow reverts to free-flow operation through the work zone for the final hour of the 24-hour period. During this last hour, only free-flow user costs are incurred. The roadway operating characteristics of the 11 p.m. to midnight period are identical to those of Period 1 discussed earlier.

Step 5. Quantify Traffic Affected by Each Cost Component

The next step is to quantify the number of vehicles involved with each cost component. For simplicity of analysis, hours of the day are aggregated into the periods with similar operating characteristics. As discussed earlier, the hours are midnight to 5 a.m., 5 to 9 a.m., 9 a.m. to 3 p.m., 3 to 8 p.m., 8 to 11 p.m., and 11 p.m. to midnight.

Table 3.10 is a modification of table 3.9. The three columns that described operating conditions (f through h) have been replaced with four columns (f) through (i) that provide information on the number of vehicles that have to (f) traverse the work zone, (g) traverse the queue, (h) stop for the queue, and (i) those that merely have to slow down. These four columns are used to identify the number of vehicles involved in the seven user cost components. The table is subdivided to cluster and subtotal the hours of the day with similar operating conditions as just discussed.

Table 3.10. Expanded work zone matrix.

AADT 134,615			Queue Rate	No. of Queued Vehicles	Number of Vehicles that			
Hour (24-Hr Clock)	Demand	Capacity			Traverse WZ	Traverse Queue	Stop 55-0-55 (mi/h)	Slow Down 55-40-55 (mi/h)
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)
0 – 1	856	2,830	-1,974	0	856	0	0	856
1 – 2	614	2,830	-2,216	0	614	0	0	614
2 – 3	509	2,830	-2,321	0	509	0	0	509
3 – 4	350	2,830	-2,480	0	350	0	0	350
4 – 5	405	2,830	-2,425	0	405	0	0	405
0 - 5					2,734	0	0	2,734
5 – 6	961	6,540	-5,579	0	0	0	0	0
6 – 7	2,540	6,540	-4,000	0	0	0	0	0
7 – 8	4,200	6,540	-2,340	0	0	0	0	0
8 – 9	3,477	6,540	-3,063	0	0	0	0	0
5 - 9				0	0	0	0	0
9 –10	3,150	2,830	320	320	2,830	2,830	3,150	0
10 –11	3,417	2,830	587	907	2,830	2,830	3,417	0
11 –12	3,639	2,830	809	1,715	2,830	2,830	3,639	0
12 –13	3,769	2,830	939	2,654	2,830	2,830	3,769	0
13 –14	3,837	2,830	1,007	3,661	2,830	2,830	3,837	0
14 –15	4,051	2,830	1,221	4,881	2,830	2,830	4,051	0
9 - 15					16,980	16,980	21,861	0
15 –16	4,725	5,454	-729	4,152	0	5,454	4,725	0
16 –17	5,849	5,454	395	4,548	0	5,454	5,849	0
17 –18	6,865	5,454	1,411	5,959	0	5,454	6,865	0
18 –19	4,289	5,454	-1,165	4,794	0	5,454	4,289	0
19 –20	2,730	5,454	-2,724	2,070	0	5,454	2,730	0
15 – 20					0	27,270	24,458	0
20 –21	2,354	2,830	-476	1,594	2,830	2,830	2,354	0
21 –22	1,998	2,830	-832	762	2,830	2,830	1,998	0
22 –23	1,610	2,830	-1,220	0	2,372**	1,767*	1,005*	605*
20 - 23					8,032	7,427	5,357	605
23 –24	1,259	2,830	-1,571	0	1,259	0	0	1,259
24 hours	67,453				29,005	51,677	51,677	4,597

* Values shown are prorated based on the portion of the hour required to clear the queue (762/1220).

** Represents hourly demand and vehicles queued from the previous hour.

Vehicles That Traverse the Work Zone – Column (f)

The traffic that traverses the work zone in column (f) is generally the traffic demand on the facility during the hours the work zone is in place. Although this is the case under free-flow operating conditions, under forced-flow conditions, the maximum number of vehicles that can traverse the work zone is limited to the capacity of the work zone, which in the example problem is 2,830 vph.

From midnight to 5 a.m., the number of vehicles traversing the work zone is the demand during the period or 2,734 vehicles. During the 9 a.m. to 3 p.m. period, the demand exceeds capacity and the number traversing the work zone is limited to the 2,830 vph capacity of the work zone, for a total of 16,980 during the period. From 3 to 8 p.m., the work zone is removed and there is no work zone to traverse.

From 8 p.m. until midnight, the number of vehicles is at first limited to the capacity of the work zone because, while hourly demand is less than capacity, there is a built-up queue on the roadway that has to dissipate. Once the queue is dissipated, some time after 10 p.m., the number of vehicles traversing the work zone area then reverts to the hourly demand. From 8 to 11 p.m., 8,032 vehicles traverse the work zone, and an additional 1,259 vehicles traverse the work zone, from 11 p.m. to midnight.

A total of 29,005 vehicles traverse the work zone over the 24-hour period, as shown at the bottom of column (f).

Vehicles That Traverse the Queue – Column (g)

All vehicles that approach the work zone when a physical queue exists must stop and work their way through the queue before entering the work zone. Traffic that arrives as the queue starts to develop will have a rather short queue, while traffic arriving when the queue is fully developed will have a much longer queue to traverse. On the other hand, vehicles arriving as the queue is dissipating will have a continually shrinking queue length to deal with. Nonetheless, all vehicles that arrive while a queue is present must traverse it.

It is important to note that because the facility is operating under a forced-flow condition, the hourly volume of vehicles traversing the queue shown in column (g) is limited to the capacity of the work zone area. This is because the only way out of the queue is through the work zone area. During the period from 9 a.m. until 3 p.m., this capacity is 2,830 vph for a total of 16,980 vehicles.

During the period from 3 until 8 p.m., even though the work zone is removed, the capacity only increases to the queue dissipation rate of 5,454 vph with 3 lanes open. During this 3 to 8 p.m. period, a total of 27,270 vehicles traverse the queue.

Finally, from 8 to 10 p.m., the hourly volume traversing the queue is once again limited to the 2,830 vph work zone capacity. During the 10 to 11 p.m. period, when the queue finally completely dissipates, the number of vehicles traversing the queue is limited to those vehicles queued at 10 p.m., plus those vehicles arriving early in the hour before the queue is completely gone. The 1,767 vehicles shown for 10 to 11 p.m. represent the prorated portion. A total of 7,427 vehicles traverse the queue during this 8 to 11 p.m. period. From 11 p.m. to midnight there is no queue and therefore no vehicles traverse it.

A total of 51,677 vehicles traverse the queue over the 24-hour period, as shown at the bottom of column (g).

Vehicles That Stop – Column (h)

Every vehicle that encounters a physical queue must come to a complete stop before traversing the queue. Over a 24-hour period, the number of vehicles that must stop is equal to the vehicles that must transverse the queue. The number of vehicles that stop each hour (column h) do not correspond directly with the number of vehicles that traverse the queue each hour (column g). The reason is that the number of vehicles that come upon a queue situation and are forced to stop is governed by demand during the hour, not by the capacity of the work zone area. Note that the number of vehicles that stop in the 10 to 11 p.m. time period (1005) is a prorated share of the vehicles that approach during the hour based on the portion of the hour (762/1220) used to totally dissipate the 762 vehicles in the queue at the start of the hour. That portion is determined by dividing the number of queued vehicles at the beginning of the hour by the queue dissipation rate during the hour (762/1220). The remaining 605 vehicles that approach during this hour (1610-1005) only have to slow down.

A total of 51,677 vehicles must stop over the 24-hour period, as shown at the bottom of column (h).

This is the same number, 51,677 vehicles, that traverse the queue shown at the bottom of column (g).

Vehicles That Slow Down – Column (i)

Column (i) reveals that, in this example, only a small portion of the daily traffic has to *just slow down* to traverse the work zone. The number of vehicles that just have to slow down prior to traversing the work zone (as opposed to coming to a complete stop) are those vehicles encountering the work zone under free-flow conditions. In the example presented here, that situation exists between midnight and 5 a.m. and some time after 10 p.m. through midnight. During these times, the hourly number of vehicles that must merely slow down equals the demand during that hour. The number of vehicles that merely slow down during the 10 to 11 p.m. period is less than traffic demand. That is because the first part of the hour is used to dissipate the last of the queue, and only the last portion of the hour operates under free-flow conditions. The 605 vehicles shown for 10 to 11 p.m. represent a prorated portion of the hourly demand.

A total of 4,597 vehicles must slow down over the 24-hour period, as shown at the bottom of column (i).

General Comment

It is important to note that, in this example, a queue develops at midmorning and remains until sometime after 10 p.m. In other situations with different traffic levels, hourly distributions, and work zone configurations, it is quite possible for a queue to develop in the morning, completely dissipate, and then reappear in the afternoon. Because the number of vehicles queued would differ, the length of the queue and associated user costs would also differ and would need to be analyzed separately.

Table 3.11 reproduces summary data on the traffic affected for each of the cost components from table 3.10.

Table 3.11. Summary traffic affected by each cost component.

Hours (24-Hr Clock)	**Number of Vehicles That**			
	Traverse WZ	**Traverse Queue**	**Stop 55-0-55**	**Slow 55-40-55**
0 - 5	2,734	0	0	2,734
5 - 9	0	0	0	0
9 – 15	16,980	16,980	21,861	0
15 – 20	0	27,270	24,458	0
20 – 23	8,032	7,427	5,357	605
23 – 24	1,259	0	0	1,259
Total	29,005	51,677	51,677	4,597

Step 6. Compute Reduced Speed Delay

Before computing actual user cost, the analyst must know the number vehicles subjected to speed changes, the number of vehicles that stop, and the delay time through both the work zone and the queue. The number of vehicles that undergo speed changes and that stop is directly related to the affected traffic, which has already been determined. Although the number of vehicles delayed through the work zone and queue area have been determined, the amount of delay can only be computed after knowing the work zone and queue area lengths and the speeds through them.

The delay time through the work zone and through the queue is computed in the same manner. In each case, the delay is determined by subtracting the time it takes to traverse either the work zone or queue length when they are present from the time it takes to travel the same distance when they are not present. Both calculations depend on the length to be traversed and the appropriate travel speeds when a work zone and/or a queue are present and when they are not.

$$\text{WZ Delay} = \frac{\text{WZ Length}}{\text{WZ Speed}} - \frac{\text{WZ Length}}{\text{Upstream Speed}}$$

$$\text{Queue Delay} = \frac{\text{Queue Length}}{\text{Queue Speed}} - \frac{\text{Queue Length}}{\text{Upstream Speed}}$$

Work zone and queue delay computations for the example problem follow:

Work Zone Reduced Speed Delay

In the example used here, the following were given:

Upstream Speed = 55 mi/h
Work Zone Speed = 40 mi/h
Work Zone Length = 5.25 miles

The work zone reduced speed delay is computed using the delay formula just discussed. Table 3.12 shows the results.

Table 3.12. Work zone reduced speed delay.

Work Zone Length (Miles)	Time at 40 mi/h (Hours)	Time at 55 mi/h (Hours)	Work Zone Delay/Veh.	
			(Hours)	(Minutes)
5.25	0.1313	0.0955	0.0358	2.1

Queue Reduced Speed Delay

Queue reduced speed delay is computed in the same manner, however, in this case the queue speed and queue length are not known. It is therefore necessary, in this case, to determine the queue speed and queue length for each of the three analysis time periods where queues exists.

Queue Speed Calculations

Speed through the queue can be determined by using the *Forced-Flow Average Speed* versus *Volume to Capacity* (V/C) ratio graphs for level of service F contained in the earlier editions of the *Highway Capacity Manual*. Using the volume through the queue and the *Free-Flow* capacity of the facility, the V/C ratio is calculated for each period and used to find the corresponding speed. Table 3.13 gives volume and capacity information along with the V/C calculations for the three different queue periods (9 to 3, 3 to 8, and 8 to 11).

Table 3.13. Queue speed.

Factors	Daily Time Period		
	9 a.m. – 3 p.m.	3 – 8 p.m.	8 – 11 p.m.
	(a)	(b)	(c)
Volume (Queue)*	2,830	5,454	2,830
Capacity (Roadway)	6,540	6,540	6,540
V/C	0.43	0.83	0.43
Speed	8 mi/h	18 mi/h	8 mi/h
* This is the volume that moves out of the queue in a 1-hour period			

Because the only way for traffic to exit the queue is through the work zone, the volume through the queue section is limited to the capacity of the work zone. The capacity of the work zone restricts the volume to 2,830 vph during the 9 a.m. to 3 p.m. and the 8 to 11 p.m. periods. The queue dissipation capacity restricts volume through the queue to 5,454 vph during the 3 to 8 p.m. period.

The capacity of the 3-lane directional facility where the queue forms (upstream of the work zone) is the same as the capacity in the 3-lane unrestricted upstream section operating in a free-flow condition just prior to the queue. That free-flow capacity was determined earlier to be 6,540 vph.

Using these values, the V/C ratios are calculated and used in conjunction with figure 3.5 to determine the respective queue speeds shown at the bottom of table 3.13.

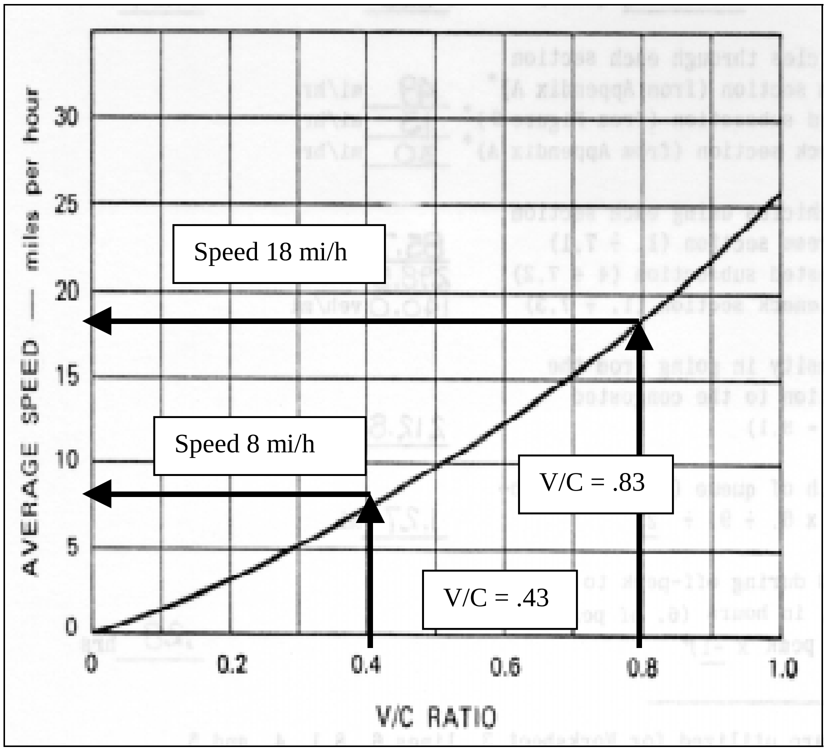


Figure 3. 5. Average speed versus V/C ratio (level of service F).

Inspection of figure 3.5 reveals that the speed through the queue for the 9 a.m. to 3 p.m. and the 8 to 11 p.m. time periods will be approximately 8 mi/h based on the computed 0.43 V/C ratio. The speed for the 3 to 8 p.m. period is approximately 18 mi/h based the computed 0.83 V/C ratio.

Queue Length Calculations

The queue length varies throughout the day with changes in directional hourly demand and capacity through the work zone section. The number of vehicles in the queue starts out small when it first forms around 9 a.m., grows to a maximum at about 6 p.m., and trails off to nothing as it totally dissipates somewhere before 11 p.m. (around 10:40 p.m.). Some vehicles face very short delays, while others face considerable delays. Figure 3.6 is a plot of the number of queued vehicles at the end of each hour. It graphically displays the growth and dissipation of the number of queued vehicles over the course of the day.

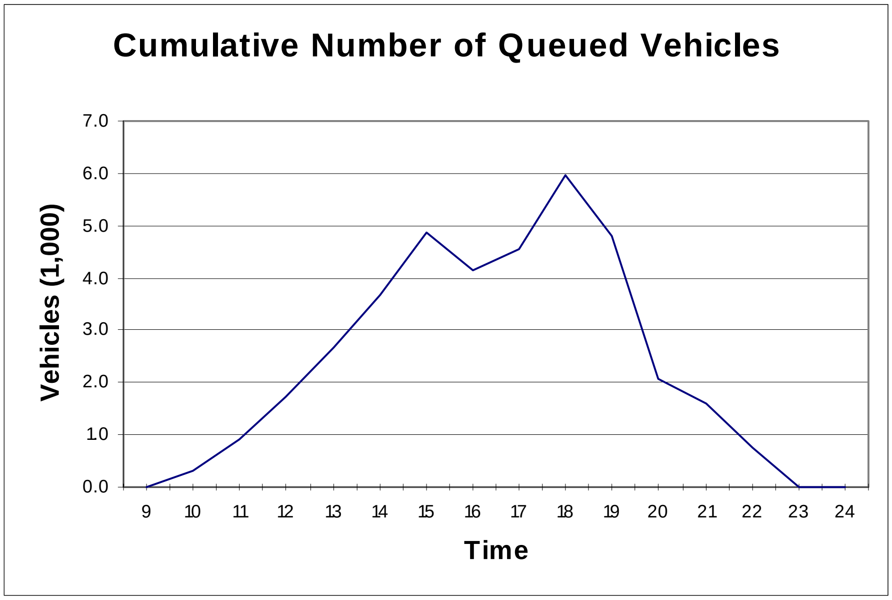


Figure 3. 6. Queued vehicle growth and dissipation over time.

Queue delay computations are generally based on the average queue length over the queue period. An average queue length can be computed for each hour that a queue exists or an overall average queue length for the entire 9 a.m. to 11 p.m. period can be developed by making some simplifying assumptions about queue growth and dissipation rates. Such assumptions, if valid, can greatly reduce the number of necessary delay time computations.

If the number of queued vehicles grew at a uniform rate to some maximum value and then dissipated at a uniform rate, the graph in figure 3.6 would be a triangle. If it were a triangle, the average queue length for the entire 9 a.m. to 11 p.m. period would simply be one-half of the maximum queue length that occurs during the day. The maximum queue length during the day occurs when the maximum number of vehicles are queued. In the example problem, this would be at 6 p.m.

Because figure 3.6 does not reflect uniform queue growth or dissipation rates, the more detailed hour-by-hour analysis is more appropriate. For the example problem, the first step is to determine the average number of vehicles queued in each hour. This is simply the arithmetic average of the number of queued vehicles at the beginning and end of each hour. The computed hourly average number of queued vehicles is plotted in figure 3.7.

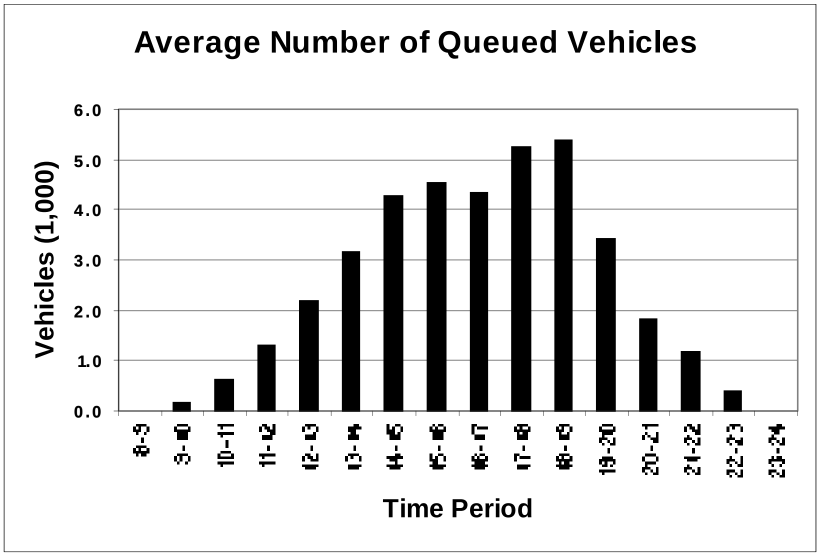


Figure 3.7. Average number of queued vehicles in each hour.

The next step is to determine the hourly queue lengths from the average number of queued vehicles for each hour.

The average number of queued vehicles in each hour is converted to average hourly queue lengths by dividing the average number of vehicles in the queue during the hour by the *change* in traffic density between the upstream free-flow section and the queue section during that hour. Traffic density is, by definition, the number of vehicles on a mile of roadway (vehicles/mile). It can be computed by dividing vehicle flow through the section (vph) by the average speed through the section (mi/h). Dividing traffic flow (vph) by traffic speed (mi/h) produces units of vehicles per mile [(vehicles/hour) / (miles/hour) = (vehicles/miles)].

Traffic densities and queue lengths are computed in table 3.14 along with the average queue lengths. Column (a) is the hour for which the queue length is being calculated. The queue and upstream volumes (columns b and c) for the hours shown in column (a) are taken from table 3.10. The queue speeds (column d) are taken directly from figure 3.5. The upstream speed (column e) was given as 55 mi/h in the problem definition.

The density in the queue (column f) is determined by dividing the queue volume in column (b) by the queue speed from column (d). The density of the upstream section (column g) is determined by dividing the upstream demand in column (c) by the upstream speed from column (e). The change in traffic density in column (h) is just the difference in the upstream and queue traffic densities (column f minus g).

The *Average Number of Queued Vehicles* (column i) is the arithmetic average of the number of vehicles queued at the beginning and end of each hour. Finally, the *Average Queue Length* in miles (column j) is computed by dividing the average number of queued vehicles for the hour (column i) by the change in traffic density (column h) during that hour.

The queue lengths in table 3.14 are grouped by analysis period (i.e., 9-15, 15-20, and 20-23) in order to determine the average queue length for the three periods. The average queue length for each of the three periods is simply the average of the average hourly queue lengths for each hour in the period.

Table 3.14. Average queue length calculations.

Time (24-Hr Clock)	Volume		Speed		Density			Average No. of Queued Vehicles
	Through Queue	Up-stream of Queue	In Queue	Up-stream of Queue	In Queue (b/d)	Up-stream of Queue (c/e)	Change (f-g)	
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)
9-10	2,830	3,150	8	55	354	57	296	160
10-11	2,830	3,417	8	55	354	62	292	613
11-12	2,830	3,639	8	55	354	66	288	1,311
12-13	2,830	3,769	8	55	354	69	285	2,185
13-14	2,830	3,837	8	55	354	70	284	3,158
14-15	2,830	4,051	8	55	354	74	280	4,271
Average for the 9-15 Period (9 a.m. to 3 p.m.)								1,950
15-16	5,454	4,725	18	55	303	86	217	4,517
16-17	5,454	5,849	18	55	303	106	197	4,350
17-18	5,454	6,865	18	55	303	125	178	5,253
18-19	5,454	4,289	18	55	303	78	225	5,376
19-20	5,454	2,730	18	55	303	50	253	3,432
Average for the 15-20 Period (3 to 8 p.m.)								4,586
20-21	2,830	2,354	8	55	354	43	311	1,832
21-22	2,830	1,998	8	55	354	36	317	1,178
22-23	2,830	1,610	8	55	354	29	324	381
Average for the 20-23 Period (8 to 11 p.m.)								1,130

For the 9 a.m. to 3 p.m. period, the average hourly queue length is 0.54 miles between 9 and 10 a.m. and continually builds to a maximum hourly average length of 15.25 miles between 2 and 3 p.m. The overall average queue length for the 9 a.m. to 3 p.m. period is 6.87 miles.

For the 3 to 8 p.m. period, the average hourly queue length starts at 20.81 miles between 3 and 4 p.m., builds to 29.48 miles between 5 and 6 p.m., and then drops back to 13.55 miles between 7 and 8 p.m. This happens despite of the fact that the work zone is not in place during this entire 3 to 8 p.m. period. The overall average queue length for the entire 5-hour period is 21.97.

Finally, during the 8 to 11 p.m. period, the average hourly queue length starts out at 5.89 miles between 8 and 9 p.m. and drops to 3.71 miles between 9 and 10 p.m. Somewhere between 10 and 11 p.m. the queue finally dissipates and the average hourly queue length for this period is 1.17 miles. The overall average queue length for the 8 to 11 p.m. period is 3.59 miles.

Queue Length Calculations – Alternate Approach

A simplified approach to calculating queue lengths entails assigning the number of queued vehicles to the available lanes and multiplying the number of vehicles per lane by an assumed average vehicle length that includes the space between vehicles. An example problem on page 6-7 of the 1994 HCM assumes an average vehicle length of 40 feet/vehicle. Using the same assumed value of 40 feet/vehicle and the average number of queued vehicles for each of the queue periods from table 3.14, the average queue length are calculated in table 3.15 using this alternate approach.

Table 3.15. Average queue length — alternative approach.

Period	**Average No. Queued Vehicles**	**No. of Lanes**	**Average No. Queued (Vehicles/Lane) (b)/(c)**	**Average Vehicle Length (feet)**	**Feet Per Mile**	**Average Queue Length (Miles) (d)x(e) / (f)**
(a)	**(b)**	**(c)**	**(d)**	**(e)**	**(f)**	**(g)**
9:00 - 15:00	1,950	3	650	40	5,280	5.0
15:00 - 20:00	4,586	3	1,529	40	5,280	11.8
20:00 - 23:00	1,130	3	377	40	5,280	2.9

The average queue lengths calculated in table 3.15 under the alternative approach are somewhat shorter for the 9 a.m. to 3 p.m. and the 8 to 11 p.m. periods (periods when the work zone is in place). This indicates the assumed vehicle length of 40 feet is about 25 percent too short. The average queue length calculated for the 3 to 8 p.m. period, when the work zone is not in place, is significantly shorter (approximately 50 percent) than the average queue length computed using the roadway density approach. During this period the queue speed is significantly higher and it is highly unlikely that the assumed 40 feet/vehicle spacing can be maintained.

Once the maximum queue lengths and the speeds through the queue for all three queue analysis periods have been determined, the average delay for vehicles that transverse each of the queues can be computed. The average delay is computed by determining the time necessary to transverse the average queue length at the forced-flow queue speed and subtracting the time it would normally take if the queue were not present. The speed without the queue present is the same as the speed in the upstream section (in this case it is given as 55 mi/h).

The average queue delay time for each of the three queue analysis periods is computed in table 3.16. The travel times shown in column (c) are computed by dividing the average queue length in column (a) by the queue speed in column (b). The travel times for column (d) are computed by dividing the average queue length in column (a) by the 55 mi/h upstream speed. The average queue delay times shown in column (e) and (f) are determined by subtracting the travel times necessary to traverse the queue length (columns c minus d). The daily hours of queue delay will be computed by multiplying the average queue delay time per vehicle by the number of affected vehicles in each period.

Table 3.16. Average queue delay time.

Period	Average Queue Length (Miles)	Queue Speed (Miles/Hour)	Time (hours)		Average Queue Delay per Vehicle	
			@ Queue Speed (a/b)	@ 55 mi/h (a/55)	Hours (c-d)	Minutes (c-d)
	(a)	(b)	(c)	(d)	(e)	(f)
9 – 15	6.87	8	0.8590	0.1249	0.7340	44.04
15 – 20	21.97	18	1.2205	0.3994	0.8211	49.26
20 - 23	3.59	8	0.4490	0.0653	0.3837	23.02

Step 7. Select and Assign VOC Rates

Table 3.17 is reproduced from chapter 2 and shows additional hours of delay and additional VOC (in August 1996 dollars) associated with stopping 1000 vehicles from a particular speed and returning them to that speed for the three vehicle classes. Different factors are provided for Passenger cars and both Single-Unit and Combination trucks. In addition, the last row of table 3.17 shows the VOC rate associated with idling while stopped.

While this table is designed to determine stopping cost, it can also be used to determine the cost and time factors associated with slowing from 55 mi/h to 40 mi/h and returning to 55 mi/h. This is accomplished by subtracting the cost and time factors for stopping associated with each

Table 3.17. Added time and vehicle running cost/1,000 stops and idling costs (Aug 96 $).

Initial Speed (mi/h)	Added Time (Hr/1,000 Stops) (Excludes Idling Time)			Added Cost ($/1,000 Stops) (Excludes Idling Time)		
	Pass. Cars	Trucks		Pass. Car	Trucks	
		Single-Unit	Combination		Single-Unit	Combination
5	1.02	0.73	1.10	2.70	9.25	33.62
10	1.51	1.47	2.27	8.83	20.72	77.49
15	2.00	2.20	3.48	15.16	33.89	129.97
20	2.49	2.93	4.76	21.74	48.40	190.06
25	2.98	3.67	6.10	28.67	63.97	256.54
30	3.46	4.40	7.56	36.10	80.23	328.21
35	3.94	5.13	9.19	44.06	96.88	403.84
40	4.42	5.87	11.09	52.70	113.97	482.21
45	4.90	6.60	13.39	62.07	130.08	562.14
50	5.37	7.33	16.37	72.31	145.96	642.41
55	5.84	8.07	20.72	83.47	160.89	721.77
60	6.31	8.80	27.94	95.70	178.98	798.99
65	6.78	9.53	NA	109.02	195.84	NA
70	7.25	NA	NA	123.61	NA	NA
75	7.71	NA	NA	139.53	NA	NA
80	8.17	NA	NA	156.85	NA	NA
Idling Cost ($/Veh-Hr)				0.6927	0.7681	0.8248

speed from one another. For the example problem, the last line of table 3.18 (columns a through c, and columns d through e, respectively) shows the delay and speed change cost factors associated with going from 55 mi/h to 40 mi/h and back to 55 mi/h.

Table 3.18. Speed change computations.

Initial Speed (mi/h)	Added Time (Hr/1,000 Stops) (Excludes Idling Time)			Added Cost ($/1,000 Stops) (Excludes Idling Time)		
	Pass. Cars	Trucks		Pass. Cars	Truck	
		Single-Unit	Combinations		Single-Unit	Combinations
	(a)	(b)	(c)	(d)	(e)	(f)
55	5.84	8.07	20.72	83.47	160.89	721.77
40	4.42	5.87	11.09	52.70	113.97	482.21
55-40-55	1.42	2.20	9.63	30.77	46.92	239.56

Step 8. Select and Assign Delay Cost Rates

The section on *Delay Cost Rates* (page 19) discusses user delay cost rates. Table 2.13 is reproduced here for convenience as table 3.19.

Table 3.19. Recommended values of travel time ($/Veh-Hr) (Aug 96 $).

Passenger Cars	Trucks	
	Single-Unit	Combinations
$ 10 to 13	$17 to 20	$ 21 to 24

While any values within the ranges shown in table 3.19 are considered reasonable, for purposes of the example problem, the *Delay* rates selected correspond to the mean values determined in table 2.7 (see page 20). The following values are used in connection with all delay cost computations in the section on *Compute User Cost Component by Vehicle Class*:

Passenger Vehicles = $ 11.58/Veh-Hr
Single-unit trucks = $ 18.54/Veh-Hr
Combination trucks = $ 22.31/Veh-Hr

Step 9. Assign Traffic to Vehicle Classes

At this point it is necessary to distribute the directional traffic affected by the various cost components to the appropriate vehicle classes for each cost component. Table 3.20 lays out the matrix used to assign the overall traffic associated with each of the user cost components to the appropriate vehicle classes.

Table 3.20. Affected traffic by vehicle class and user cost component.

Cost Component	Affected Vehicles Mixed Flow	Passenger Vehicles 89%	Trucks		Total
			Single-Unit 6%	Combos 5%	
	(a)	(b)	(c)	(d)	(e)
Speed Change (55-40-55)	4,597	4,092	276	230	4,597
Traverse WZ	29,005	25,814	1,740	1,450	29,005
Stopping (55-0-55)	51,677	45,993	3,101	2,584	51,677
Queue Delay$_1$ (9 –15)	16,980	15,112	1,019	849	16,980
Queue Delay$_2$ (15-20)	27,270	24,270	1,636	1,364	27,270
Queue Delay$_3$ (20-23)	7,427	6,610	446	371	7,427

Continuing with the example problem, column (a) of table 3.20 lists the total daily traffic for each user cost component. The volumes listed in column (b), (c), and (d) reflect the distribution of the overall traffic for each of the cost components to the appropriate vehicle classes. The header for each column lists vehicle class distribution factors used. These percentages were developed for the example problem in the *Project Future Year Traffic Demand* section (page 43). They reflect the projected distributions in 1999 (the year of the work zone will be established) based on the differential growth rates assigned to passenger vehicles and trucks. Column (e) is the sum of columns (b), (c), and (d) and is just a mathematical check to ensure that the traffic assigned to the vehicle classes totals back to the original traffic volume.

Step 10. Compute User Cost Components by Vehicle Class

Daily user costs by vehicle class for each cost component are computed by multiplying the affected traffic by the appropriate unit cost rates (either VOC or delay) for the various components. The individual costs are computed in tables 3.21 through 3.27.

The added VOC rates used in column (b) of table 3.21 are from the bottom line of columns (d) through (f) in table 3.18, while the rates used in column (b) of table 3.22 are from the bottom line of columns (a) through (c) of table 3.18.

The *Idle* cost rates in column (c) of table 3.26 are taken from the bottom line of table 3.17.

Table 3.21. User cost component # 1— speed change VOC (55-40-55 mi/h).

Vehicle Class	Affected Vehicles	Added VOC (55-40-55) $/1,000 Vehicles	Cost ($) per day	Total Costs ($) (60 Days)
	(a)	(b)	(c)	(d)
Passenger Cars	4,092	30.77	126	7,554
Single-Unit Truck	276	46.92	13	777
Combination Truck	230	239.56	55	3,304
Total Speed Change VOC	4,597		194	11,634

Table 3.22. User cost component # 2 — speed change delay cost (55-40-55 mi/h).

Vehicle Class	Affected Vehicles	Added Time (55-40-55) Hrs/1,000 Veh	Delay Cost Rate ($) (per Veh-Hr)	Cost ($) per day	Total Costs ($) (60 Days)
	(a)	(b)	(c)	(d)	(e)
Passenger Cars	4,092	1.42	$11.58	67	4,037
Single-Unit Truck	276	2.20	$18.54	11	675
Combination Truck	230	9.63	$22.31	49	2,963
Total Speed Change Delay	4,597			128	7,675

Table 3.23. User cost component # 3 — work zone reduced speed delay cost.

Vehicle Class	Affected Vehicles	Added Time (55-0-55) (Hrs/1,000 Veh)	Cost ($) (per Veh-Hr)	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)	(e)
Passenger Cars	45,993	5.84	$11.58	3,110	186,621
Single-Unit Truck	3,101	8.07	$18.54	464	27,835
Combination Truck	2,584	20.72	$22.31	1,194	71,665
Total Stopping Delay	51,677			4,769	286,121

Table 3.24. User cost component # 4 — stopping VOC (55-0-55mi/h).

Vehicle Class	Affected Vehicles	Added VOC (55-0-55) Hrs/1,000 Veh	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)
Passenger Cars	45,993	83.47	3,839	230,341
Single-Unit Truck	3,101	160.89	499	29,932
Combination Truck	2,584	178.98	462	27,748
Total Stopping VOC	51,677		4,800	288,020

Table 3.25. User cost component # 5 — stopping delay cost (55-0-55 mi/h).

Vehicle Class	Affected Vehicles	Added Time (55-0-55) (Hrs/1,000 Veh)	Cost ($) (per Veh-Hr)	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)	(e)
Passenger Cars	45,993	5.84	$11.58	3,110	186,621
Single-Unit Truck	3,101	8.07	$18.54	464	27,835
Combination Truck	2,584	20.72	$22.31	1,194	71,665
Total Stopping Delay	51,677			4,769	286,121

Table 3.26. User cost component # 6 — idling VOC.

Vehicle Class	Queue Period	Affected Vehicles	Added Time (Hours)	Idle VOC Rates ($/1000Veh-Hr)	Costs ($) per day	Total Cost (60 days)
		(a)	(b)	(c)	(d)	(e)
Passenger Cars	9 a.m. to 3 p.m.	15,112	0.7340	692.70	7,684	461,038
	3 p.m. to 8 p.m.	24,270	0.8211		13,804	828,227
	8 p.m. to 11 p.m.	6,610	0.3837		1,757	105,422
	Subtotal	45,992	-	-	23,245	1,394,687
Single-Unit Trucks	9 a.m. to 3 p.m.	1,019	0.7340	768.10	574	34,464
	3 p.m. to 8 p.m.	1,636	0.8211		1,032	61,913
	8 p.m. to 11 p.m.	446	0.3837		131	7,881
	Subtotal	3,101	-	-	1,738	104,258
Combination Trucks	9 a.m. to 3 p.m.	849	0.7340	824.80	514	30,840
	3 p.m. to 8 p.m.	1,364	0.8211		923	55,403
	8 p.m. to 11 p.m.	371	0.3837		118	7,052
	Subtotal	2,584	-	-	1,555	93,295
Total Idling VOC						1,592,241

Table 3.27. User cost component # 7 — queue reduced speed delay cost.

Vehicle Class	Queue Period	Affected Vehicles	Added Time (Hours)	Delay Cost Rate ($) (per Veh-Hr)	Costs ($) per day	Total Cost (60 days)
		(a)	(b)	(c)	(d)	(e)
Passenger Cars	9 a.m. to 3 p.m.	15,112	0.7340	$11.58	128,454	7,707,264
	3 p.m. to 8 p.m.	24,270	0.8211		230,760	13,845,629
	8 p.m. to 11 p.m.	6,610	0.3837		29,373	1,762,360
	Subtotal	45,992	-	-	388,588	23,315,253
Single-Unit Trucks	9 a.m. to 3 p.m.	1,019	0.7340	$18.54	13,865	831,884
	3 p.m. to 8 p.m.	1,636	0.8211		24,907	1,494,428
	8 p.m. to 11 p.m.	446	0.3837		3,170	190,220
	Subtotal	3,101	-	-	41,942	2,516,533
Combination Trucks	9 a.m. to 3 p.m.	849	0.7340	$22.31	13,903	834,202
	3 p.m. to 8 p.m.	1,364	0.8211		24,977	1,498,593
	8 p.m. to 11 p.m.	371	0.3837		3,179	190,750
	Subtotal	2,584	-	-	42,059	2,523,546
Total Queue Speed Delay						28,355,331

Step 11. Sum Total Work Zone User Costs

Table 3.28 shows a master summary of all costs, and table 3.29 shows the percent distributions of those costs. The first three cost components in tables 3.28 and 3.29 represent the cost associated with free-flow, while the remaining four cost components represent the forced-flow queuing costs.

Examination of tables 3.28 and 3.29 immediately reveals that the high user costs are not a LCCA problem, but are a traffic control problem. Further inspection reveals that more than 90 percent of the user costs result from the queue delay component. An additional 5 percent is

Table 3.28. Master summary — total (60 day) work zone user cost (Aug 96 $).

User Cost Component		Passenger Cars	Trucks		Totals ($)
			Single-Unit	Combination	
		(a)	(b)	(c)	(d)
1 Speed Change VOC		7,554	777	3,304	11,634
2 Speed Change Delay		4,037	675	2,963	7,675
3 WZ-Reduced Speed Delay		642,012	69,296	69,489	780,796
4 Stopping VOC		230,341	29,932	27,748	288,020
5 Stopping Delay		186,621	27,835	71,665	286,121
6 Queue Idling VOC	9 a.m. to 3 p.m.	461,038	34,464	30,840	1,592,241
	3 p.m. to 8 p.m.	828,227	61,913	55,403	
	8 p.m. to 11 p.m.	105,422	7,881	7,052	
	Subtotal	1,394,687	104,258	93,295	
7 Queue Speed Delay	9 a.m. to 3 p.m.	7,707,264	831,884	834,202	28,355,331
	3 p.m. to 8 p.m.	13,845,629	1,494,428	1,498,593	
	8 p.m. to 11 p.m.	1,762,360	190,220	190,750	
	Subtotal	23,315,253	2,516,533	2,523,546	
Grand Totals		25,780,505	2,749,304	2,792,010	31,321,819
Grand Totals %		82.3%	8.8%	8.9%	100.0%

Table 3.29. Master summary — work zone user cost distribution (%).

User Cost Component	Pass Cars	SU	Combination	Totals
1 Speed Change VOC	0.02	0.00	0.01	0.03
2 Speed Change Delay	0.01	0.00	0.01	0.02
3 WZ-Reduced Speed Delay	2.05	0.22	0.22	2.49
4 Stopping VOC	0.74	0.10	0.09	0.93
5 Stopping Delay	0.60	0.09	0.23	0.92
6 Queue Idling VOC	4.45	0.33	0.30	5.08
7 Queue Speed Delay	74.44	8.03	8.06	90.53
Grand Totals	82.31	8.78	8.92	100.00

associated with the queue idling costs and another 2 percent is associated with queue stopping VOC and delay. Therefore, approximately 97 percent of the user costs can be avoided by not allowing the queues to develop in the first place. In the example problem, the queuing situation could be drastically reduced, if not completely avoided, if work zone operations could be limited to evening work between 7 p.m. to 7 a.m. By limiting the contractor to evening work hours only, the queue cost in the 9 a.m. to 3 p.m. period would be completely eliminated *and* the evening rush hour would not have to deal with the built-up queue from the midday work zone!

The contractor's productivity rate would suffer dramatically during the midday use of the facility because the contractor's delivery vehicles would have to deal with the same delays as the general traffic stream. It is therefore not a large penalty on the contractor to be unable to work during midday.

Other alternatives to lowering the work zone-related user costs include adding capacity prior to the development of large future traffic demands, accelerating contractor production to reduce the overall duration the work zone is in place, and limiting the overall frequency of rehabilitation activities.

While the numbers may appear unreasonably large, they are not out of line when compared to user costs developed for highway facilities damaged by the North Ridge, California, earthquake. Table 3.30 shows estimates of the daily user costs associated with having the damaged facilities out of service.

Table 3.30. Average weekday delay — North Ridge earthquake.

Route	Delay Hours		Delay Cost		Fuel Costs ($)	Total Costs ($)
	Car	Truck	Car @ $6/hr	Truck @ $19.20/hr		
I-5	51,650	4,400	310,000	85,000	40,800	436,000
I-10	135,100	3,532	811,000	68,000	110,000	990,000
SR-118	31,630	1,235	189,800	23,710	24,600	238,100
Total	218,380	9,167	1,310,800	176,710	175,400	1,664,100

Source: Wesemann, Hamilton, Tabaie, and Bare, 1995.
Note: 30 percent of companies severely affected; average shipping cost increased by 8 percent.

Note: In the example problem, congestion delay is all work zone related. Prior to establishing the work zone, the highway facility could handle every hourly demand. In other cases, this may not be true—the facility may be congested on a regular basis without work zones. In such cases, *routine congestion* costs must be subtracted from the work zone-related congestion.

Step 12. Address Circuity and Delay Costs

The final step in calculating user costs is to address circuity and crash costs. The following sections address these costs.

CIRCUITY

Circuity is a term used to describe the additional mileage that users travel, either voluntarily or involuntarily, on a detour to avoid a highway work zone. If traffic is forced to detour, circuity costs are the:

1. Full VOC costs ($0.31 per mile) for passenger cars times the excess distance the detour imposes.
2. Appropriate $/hour delay rates times the excess detour time.
3. Difference in crash rates times the exposure rate and cost per crash.

If no formal detour is established and circuity is the result of voluntary self-imposed diversions, then a consumer surplus approach must be employed.

Example Circuity Delay Calculations

Problem Statement

A 5-mile rural section of Parkway between interchanges will be closed for 90 days to reconstruct a bridge taken out by flash flooding. The reconstruction will require traffic to be detoured to a 10-mile section of rural 2-lane minor arterial highway. Capacity on the minor arterial is currently underutilized and congestion is not expected to be a major problem. AADT on the parkway is 10,000 vehicles per day.

- Circuity Delay Costs

(Detour) 10 miles @ 30 mi/h = 0.333 hours per vehicle
– (Parkway) 5 miles @ 50 mi/h = 0.100 hours per vehicle
Additional hours per vehicle = 0.233

Additional Delay Cost =
0.233 hrs/veh x 90 days x 10,000 AADT x $11.50/veh-hr = $2,411,550

- Circuity VOC (Assume a VOC rate of $0.30/mile on the Parkway and $0.35/mile on the detour)

VOC (Detour) = 10 miles x 10,000 vpd x 90 days x $0.35 / VMT = $ 3,150,000
– VOC (Parkway) = 5 miles x 10,000 vpd x 90 days x $0.30 / VMT = $ 1,350,000
Additional VOC (Circuity) = $ 1,800,000

- Total Circuity Costs $ 2,411,550 Additional Delay Costs

$ 1,800,000 Additional VOC Costs

$ 4,211,550 Total Circuity Cost

CRASH COSTS

General

The highway safety community has replaced the term *accident* with the term *crash* because the term accident implies that they are unavoidable. In reality, highway crashes to a large extent *are* avoidable. This *Interim Technical Bulletin* uses the term *crash costs* to describe what was previously called *accident costs.*

Crash costs associated with work zones and work zone-generated circuitous travel are a function of the generally higher crash rates in work zones and on alternate/detour routes than on the primary facility in the absence of work zones. Crash rates are based on the number of crashes as a function of exposure, typically vehicle miles of travel (VMT). Crash rates are commonly specified as crashes per 100 million vehicle miles of travel (100 M VMT).

Overall crash rates for the various functional classes of roadway are fairly well-established. Crash rates for work zones, however, are not. While there is a limited amount of work zone crash history data, the validity of the data used to compute the crash rates is sometimes suspect. Sometimes, crashes that occur in work zone-generated queues are not classified as *work zone* crashes. Even more importantly, most of the time it's difficult to accurately quantify the work zone exposure rate (i.e., the length of the work zone and number of hours a day, and the number of days the work zone and resultant queues are in place). Further, the crash rate, while significantly higher in work zones than nonwork zones, is sometimes still low enough that there aren't any crashes in a given work zone because the exposure period is just too short to allow for statistically valid results. Finally, the problem is compounded by the fact that work zones differ significantly in the way they treat maintenance of traffic. For example, some use permanent barriers, while others use cones or drums; some narrow lanes, while others maintain lane width and shoulders. Although there appears to be a general rule of thumb indicating that crash rates in work zones are about three times the normal rate for the facility, there does not appear to be much statistically significant research data to support this rule of thumb.

With these limitations on the availability and validity of crash rates in mind, crash costs can be developed by multiplying the unit cost per crash, by the differential crash rates between work zones and nonwork zones, by the vehicle miles of travel during the duration of the work zone. The duration includes both the time the work zone is in place and the time that queues are present. Vehicle miles of travel can be estimated by multiplying the percent of the facility's AADT that will be affected by the work zone and resultant queues, by the length of the work zone and queue, and the number of days the work zone will be in place.

Overall Crash Rates

Crash costs on detours and alternate routes used by diverted traffic are generally easier to calculate because crash rates on the primary facility without a work zone and on alternate routes are generally better known. However, even when the crash rates are better known, the exposure rate may still be difficult to determine as it requires some indication or estimate of diversion to alternate routes when no formal detours are established.

Crash rates by crash type for the various roadway functional classes are computed by using crash and VMT data contained in the FHWA's most current annual *Highway Statistics* report.(17) Table 3.31 contains 1995 data on crash fatalities and nonfatal injuries listed in the FWHA's *1995 Highway Statistics*. Table 3.32 contains *1995 Highway Statistics* data on VMT. The data in these two tables are used to generate the crash injury rates contained in table 3.33. The overall crash injury rates shown in table 3.33 are people injured per 100 M VMT and do not specifically represent work zone injury rates.

Table 3.31. 1995 people injured in motor vehicle crashes by functional class.

Facility Type	Rural		Urban	
	Fatalities	Nonfatal Injuries	Fatalities	Nonfatal Injuries
(a)	(b)	(c)	(d)	(e)
Interstate	2,340	55,924	2,110	253,823
Other Freeways/Expressways	-	-	1,290	86,676
Other Principal Arterial	4,498	120,271	5,868	773,446
Minor Arterial	4,300	166,241	3,755	515,586
Major Collector	5,111	187,648	-	-
Minor Collector	1,565	60,143	-	-
Collectors	-	-	2,568	218,837
Local	3,910	236,441	4,455	601,383
Total	21,724	826,668	20,046	2,449,751

Source: *1995 Highway Statistics:* tables Fl-220 and Fl 221.

Table 3.32. 1995 vehicles miles of travel (millions).

Facility Type	Rural	Urban	Total
(a)	(b)	(c)	(d)
Interstate	223,382	341,528	564,910
Other Freeway/Expressway	-	151,560	151,560
Other Principal Arterial	215,567	370,338	585,905
Minor Arterial	153,028	293,272	446,300
Major Collector	186,212	-	186,212
Minor Collector	49,936	-	49,936
Collector	-	126,929	126,929
Local	105,164	205,907	311,071
Total	933,289	1,489,534	2,422,823

Source: *1995 Highway Statistics:* table VM-202.

Table 3.33. Crash injury rates (people injured per 100 M VMT)

Facility Type	Rural		Urban	
	Fatalities	Nonfatal Injury	Fatalities	Nonfatal Injuries
(a)	(b)	(c)	(d)	(e)
Interstate	1.0	25.0	0.6	74.3
Other Freeways/Expressways	-	-	0.9	57.2
Other Principal Arterial	2.1	55.8	1.6	208.8
Minor Arterial	2.8	108.6	1.3	175.8
Major Collector	2.7	100.8	-	-
Minor Collector	3.1	120.4	-	-
Collectors	-	-	2.0	172.4
Local	3.7	224.8	2.2	292.1

Source: Computed from *1995 Highway Statistics* data.

Table 3.33 shows the crash injury rates for rural and urban areas. These were computed by dividing the number of rural injuries (columns b and c) and urban injuries (columns d and e) in table 3.31 by the appropriate VMT contained in columns (b) and (c) of table 3.32. The crash injury rates listed in table 3.33 are the number of fatalities (column b and d) or the number of nonfatal injuries (columns c and e) per 100 M VMT.

Work Zone Crash Rates

Limited information on work zone crashes is contained in the FHWA's *Construction Cost and Safety Impacts of Work Zone Traffic Control Strategies*.(18) The study evaluated the differences in Single Lane Closure (SLC) and Two-Lane Two-Way Operation (TLTWO … cross over) traffic control strategies on rural 4-lane divided highways. The report includes crash data from 48 construction projects in 11 States. The traffic volumes on the routes were relatively low, generally within a 10,000 to 30,000 ADT range.

The report concluded that there were no significant differences in overall crash rates between the two traffic control alternatives for the sites and volumes studied. It further concluded that there was no significant difference in total crashes rates between before versus during construction for all projects. However, the report did conclude that the severity (the number of fatal and nonfatal injury crashes) increased significantly during construction periods for both the SLC and TLTWO traffic control strategies.

The 1996 Fatality Analysis Reporting System (FARS) data supports this finding. Column (a) of table 3.34 lists ranges of the percent of total motor vehicle crash fatalities that occur in work zones, while column (b) lists the number of States that fall within a range.(19)

Table 3.34. 1996 work zone motor vehicle crash fatalities as a percent of all fatalities.

Work Zone Fatality Ratio Ranges (%)	No. of States	
(a)	(b)	(c)
0%	3	3
0.01% – 0.49%	1	9
0.50% – 0.99%	8	
1.00% – 1.49%	16	23
1.50% – 1.99%	7	
2.00% – 2.49%	6	11
2.50% - 2.99%	5	
3.00% - 3.49%	1	2
3.50% - 3.99%	1	
4.00% - 4.49%	3	3

Inspection reveals that most SHAs are experiencing a disproportionate percentage of fatalities (more that 1 percent) occurring in work zones relative to the VMT that takes place in work zones.

Construction Cost and Safety Impacts of Work Zone Traffic Control Strategies also includes crash rates on the study routes before and during the time that work zones were established. The crash rates provided in the report are broken out by type of work zone, type of construction work, and type of traffic control used. However, because of the limited number of projects in the study, the statistical validity of such highly disaggregated analysis is questionable.

As a result, tables 3.35 and 3.36 present only the overall work zone crash rate data for the two work zone traffic control strategies contained in the report.

Columns (e) and (f) in tables 3.35 and 3.36 contain a computed crash rate in the form of the annual number crashes per mile of highway per 10,000 ADT for the time periods before and during which the work zones are in place. Column (g) shows the difference. The units used in the original report (annual number crashes per mile of highway per 10,000 ADT) are converted to more commonly used units (crashes per 100 M VMT) in columns (h) through (j). Inspection of tables 3.35 and 3.36 reveals some projects had lower crash rates during the period of construction than they had during the period before

Table 3.35. Crash rates on SLC.

State	Route	Length (SLC)	ADT (1,000)	Annual Crashes per Mile per 10 K ADT			Crashes per 100 Million VMT		
				Before	During	Change	Before	During	Change
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
AZ	I-10	14.1	8.0	6.881	5.802	-1.079	183.0	159.0	-24.1
KY	I-75	8.0	26.0	1.414	0.877	-0.536	38.7	24.0	-14.7
AZ	I-40	6.2	8.8	2.478	2.058	-0.419	67.9	56.4	-11.5
LA	I-20	6.1	23.5	1.133	0.850	-0.283	31.0	23.3	-7.8
NC	I-95	11.6	20.0	1.893	1.854	-0.039	59.1	50.8	-1.1
LA	I-12	14.8	21.6	1.268	1.250	-0.018	34.7	34.2	-0.5
MI	I-196	3.0	11.4	0.000	0.000	0.000	0.0	0.0	0.0
FL	I-295	7.5	26.0	3.336	3.423	0.086	91.4	93.8	2.4
OR	I-84	16.9	12.4	1.044	1.160	0.116	28.6	31.8	3.2
NC	I-40	7.8	35.0	2.419	2.569	0.150	66.3	70.4	4.1
AZ	I-10	3.3	33.0	4.506	4.692	0.186	123.5	128.5	5.1
OR	I-5	15.5	20.6	0.297	0.787	0.490	8.1	21.6	13.4
NC	I-85	7.4	30.0	2.023	2.722	0.699	55.4	74.6	19.2
OH	I-75	7.0	25.4	1.754	2.517	0.763	48.1	69.0	20.9
KY	SR-114	14.4	7.6	2.298	3.117	0.819	63.0	87.0	24.1
NC	I-85	13.1	41.3	1.803	2.667	0.864	49.4	73.1	23.7
OR	I-84	18.4	5.6	2.053	2.980	0.927	56.2	81.6	25.4
NC	I-77	9.9	32.0	1.328	2.739	1.411	36.4	75.0	38.7
LA	I-10	11.0	24.1	1.659	3.094	1.435	45.5	84.8	39.3
WV	I-79	4.2	6.2	2.657	4.971	2.313	72.8	136.2	63.4
AZ	I-8	6.1	5.9	0.939	3.381	2.442	25.7	92.6	66.9
FL	I-295	4.8	20.0	4.999	7.604	2.605	137.0	208.3	71.4
UT	I-15	16.3	4.5	3.022	5.768	2.747	82.8	158.0	75.2
NC	I-40	3.4	30.0	1.535	4.435	2.900	42.1	121.5	79.5
MI	I-96	9.9	24.5	4.317	7.935	3.618	118.3	217.4	99.1
WV	I-64	2.7	19.0	1.587	9.522	7.935	43.5	260.1	217.4
Total		243.3	522.3						
Average				2.256	3.414	1.159	61.9	93.6	32.0
STD				1.525	2.372	1.840	41.10	64.9	50.10

Source: FHWA-RD-89-210.

Table 3.36. Crash rates on TLTWO.

State	Route	Length (SLC)	ADT (1,000)	Annual Crashes per Mile per 10 K ADT			Crashes per 100 Million VMT		
				Before	During	Change	Before	During	Change
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
LA	I-20	7.2	23.9	6.607	2.332	-4.285	181.1	61.2	-119.8
UT	I-15	8.8	5.2	5.447	2.815	-2.631	149.2	77.1	-72.1
WV	I-64	2.1	27.0	5.495	2.931	-2.565	150.5	80.3	-70.2
LA	US-190	2.4	16.0	9.505	7.097	-2.408	260.4	194.4	-66.0
MI	I-96	12.0	12.8	5.104	3.062	-2.042	139.8	83.9	-55.9
FL	SR-95	2.8	4.9	5.173	4.803	-0.369	141.7	131.6	-10.1
MI	I-94	10.0	29.0	2.884	2.535	-0.350	79.0	69.5	-9.6
KY	WKP	5.5	4.2	2.048	1.756	-0.293	56.1	48.1	-8.0
UT	I-84	10.7	3.8	3.265	3.019	-0.246	89.5	82.7	-6.7
AZ	I-10	2.2	12.0	2.496	2.304	-0.192	68.4	63.1	-5.3
LA	I-20	8.8	13.5	1.703	1.606	-0.097	46.7	44.0	-2.7
OR	I-5	13.2	22.6	0.978	1.043	0.065	26.8	28.6	1.8
LA	I-20	4.7	27.6	1.590	1.877	0.287	43.6	51.4	7.9
MI	I-94	10.0	18.5	3.886	4.185	0.299	106.5	114.7	8.2
LA	I-59	6.6	13.0	3.314	3.615	0.301	90.8	99.0	8.2
NC	I-40	16.9	25.0	0.980	1.700	0.720	26.8	46.6	19.7
KY	I-75	4.0	23.0	1.469	2.498	1.029	40.2	68.4	28.2
OR	I-5	7.1	24.2	0.712	1.780	1.068	19.5	48.8	29.3
NC	I-40	11.7	17.0	2.050	3.415	1.365	56.2	93.6	37.4
WV	I-77	2.6	9.3	2.516	4.193	1.677	68.9	114.9	45.9
NC	US-1	1.8	15.0	2.674	4.457	1.783	73.3	122.1	48.8
MI	I-69	6.1	15.5	1.397	4.239	2.841	38.3	116.1	77.9
Total		157.1	363.0						
Average				3.241	3.057	-0.184	88.8	83.6	-5.1
STD				2.191	1.384	1.697	60.0	38.0	46.8

Source: FHWA-RD-89-210.

the work zone was established. Further, from the bottom lines of the tables, the overall average crash rate on SLC was slightly (though not significantly) higher during work zone operations, while the overall average crash rate on TLTWO was slightly (though again not significantly) lower during work zone operations.

Work zone crash rate data is also included in *An Evaluation of Lane Closure Strategies for Interstate Work Zone*.(20) Data contained in this report was based on the study of 26 Interstate 4R Projects in Indiana. Tables 3.37 and 3.38 provide data on overall and injury crash rates.

Note that the rates in tables 3.37 and 3.38 are per 10 M VMT.

Table 3.37. Average overall crash rates.

Work Zone Type	Lanes in Each Direction	Mean and Standard Deviation	Crash Rates per 10 M VMT		
			Without Work Zone	With Work Zone	Change
Cross Over	2	Mean	6.0329	8.0431	2.0102
		Standard Deviation	1.6842	3.6017	3.2005
Partial Lane Closure	2	Mean	5.5916	7.4528	1.8612
		Standard Deviation	1.4645	3.1398	3.3354
Cross Over	3	Mean	5.8278	9.3544	3.5266
		Standard Deviation	1.2350	5.9645	5.6871
Partial Lane Closure	3	Mean	7.5166	10.1006	2.5840
		Standard Deviation	1.6422	2.6940	3.4964

Source: FHWA/JHRP-95/1

Table 3.38. Average fatal and nonfatal injury crash rates.

Work Zone Type	Lanes in Each Direction	Mean and Standard Deviation	Injury Crash Rates per 10 M VMT		
			Without Work Zone	With Work Zone	Change
Cross Over	2	Mean	1.1289	2.0746	0.9457
		Standard Deviation	0.5376	1.9380	2.1879
Partial Lane Closure	2	Mean	1.0969	2.0311	0.9342
		Standard Deviation	0.4252	1.3405	1.2684
Cross Over	3	Mean	1.5885	2.6367	1.0482
		Standard Deviation	0.3961	1.5320	1.3851
Partial Lane Closure	3	Mean	1.7641	2.1128	0.3487
		Standard Deviation	0.2829	1.0574	1.1565

Source: FHWA/JHRP-95/1

Example Crash Cost Calculations

The following example problem is used to lay out a rational approach to calculating the additional crash costs associated with a work zone-generated detour.

Problem Statement

A 5-mile rural section of Parkway between interchanges will be closed for 90 days to reconstruct a bridge taken out by flash flooding. The reconstruction will require traffic to be detoured to a 10-mile section of rural 2-lane minor arterial highway. Capacity on the minor arterial is currently underutilized and congestion is not expected to be a major problem. AADT on the parkway is 10,000 vehicles per day.

Solution

Table 3.39 shows the matrix used to calculate the differential crash costs. Column (c) lists the crash rates for the facility and crash types listed in columns (a) and (b). The rates themselves are taken from columns (b) and (c) of table 3.33. The crash fatality rate is 2.8 fatalities per 100 M VMT on rural 2-lane minor arterial highways and 2.1 on rural parkways (other principal arterial routes). The injury crash rates per 100 M VMT is 108.6 on rural 2-lane minor arterial highways and 55.8 on rural parkways (other principal arterial routes).

Table 3.39. Crash cost calculation matrix.

Facility	**Crash Rate**		**Exposure**				**No. of Crashes (c)x(g)**	**Crash Cost Rate**	**Crash Costs ($) (h)x(i)**
	Type	**Crash/ 100 M VMT**	**Vehicles per day**	**No. of Days**	**Miles**	**VMT (100 M) (d)x(e)x(f)**			
(a)	**(b)**	**(c)**	**(d)**	**(e)**	**(f)**	**(g)**	**(h)**	**(i)**	**(j)**
Parkway	Fatality	2.1	10,000	90	5	0.045	0.0945	1,240,000	117,180
	Injury	55.8	10,000	90	5	0.045	2.5110	27,800	69,806
Subtotal	All								186,986
Detour	Fatality	2.8	10,000	90	10	0.090	0.2520	1,240,000	312,480
	Injury	108.6	10,000	90	10	0.090	9.7740	27,800	271,717
Subtotal	All								584,197
Change									397,211

In addition to the higher fatality and nonfatal injury crash rates, detoured and diversionary traffic on average have a greater crash exposure because of the generally greater travel distances. Column (g) of table 3.39 lists the exposure for the two alternative routes in units of 100 M VMT. The values listed were computed by multiplying the ADT (column d) by the number of days (column e) by the length of each alternative route (column f) and dividing the answer by 100 million.

The actual number of crashes expected (column h) are calculated by multiplying the crash rates in column (c) by the exposure in column (g).

Finally, the crash cost listed in column (j) are computed by multiplying the expected number of crashes in column (h) by the crash cost rates in column (i). The crash cost rates used in column (i) were selected from the updated MicroBENCOST default crash cost rates discussed earlier in *Crash Cost Rates* on page 23.

Further information on work zone crash rates is available in the participant's handbook for National Highway Institute Course # 38003, *Design and Operation of Work Zone Traffic Control.*[(21)] Pages 2-7 discuss *Before and After* work zone crash rates. Results from Virginia and Texas are included.

Virginia data for 1991 indicate an increase of 57 percent on multilane highways and 168 percent on 2-lane urban highways with the variation being a function of traffic, geometry, and environment. In Texas, 1984 to 1988 data show freeway construction crashes increased by 28.7 percent on the main line and 2.4 percent on frontage roads. It also shows a 37.4 percent increase at nighttime compared to 24.4 percent during the daytime. The data also suggest that the average changes in severe mainline crash rates were consistent from project to project, while other mainline crash categories varied significantly from site to site.

LCCA Microcomputer Programs

Microcomputer software programs such as *MicroBENCOST* and *QueWZ* are available for conducting LCCA on routine pavement rehabilitation projects. The simplified procedures outlined in this chapter are not likely to be sufficient when attempting to analyze user cost on highly complex urban freeways. In such cases, the analyst would do better to use the battery of urban planning models used by the major metropolitan planning organizations (MPOs). Traffic assignment models, run on the network level can provide better estimates of traffic diversions from the route in question and the effect of such diversions of traffic flow on alternate routes. These models can be run to determine the network-level changes in vehicle miles of travel (VMT) and hours of traffic delay with and without the work zone in place. When network level changes in VMT and Delay are known, user-cost calculations become much simpler.

CHAPTER 4. RISK ANALYSIS APPROACH

This chapter introduces a probabilistic-based risk analysis approach to LCCA in pavement design. It introduces the Monte Carlo simulation technique as the method of choice in the treatment of uncertain LCCA variables. It explores the concept of risk and examines some of the limitations associated with the current deterministic approach. While this chapter focuses on the use of risk analysis in a pavement design setting, the principles and techniques put forth offer potential in other areas where uncertainty is an important consideration in the decision-making process.

DEFINING RISK

The concept of risk comes from the uncertainty associated with future events — the inability to know what the future will bring in response to a given action today. Risk can be subjective or objective. *Subjective risk* is based on personal perception — intuitively deciding how risky a situation may be. For example, many people may feel that flying is more risky than driving. This perception of risk may be related to the consequences of failure, as well as the ability (or inability) to control the situation. *Objective risk* is based on theory, experiment, or observation. Often the facts of the situation may dispute intuitive feelings. For example, in 1996 there were 1,070 aviation fatalities; in the same year there were also 40,676 highway fatalities. Because individuals' perceptions of risk vary, decisions incorporating risk management concepts will depend to a large extent on the decision maker's tolerance for risk.

DEFINING RISK ANALYSIS

Risk analysis is concerned with three basic questions about risk: 1) What can happen? 2) How likely is it to happen? 3) What are the consequences of its happening? Risk analysis answers these questions by combining probabilistic descriptions of uncertain input parameters with computer simulation to characterize the risk associated with future outcomes. It exposes areas of uncertainty typically hidden in the traditional deterministic approach to LCCA, and it allows the decision maker to weigh the probability of an outcome actually occurring.

THE NEED FOR RISK ANALYSIS

Many analytical models treat input variables as discrete fixed values, as if the values were certain. In fact, the majority of input variables are uncertain. Economic models used in a typical LCCA are no exception. In conducting LCCA, it is important to be aware of the inherent uncertainty surrounding the variables used as inputs into the analysis. Uncertainty results from the assumptions, estimates, and projections made in conducting the analysis. Table 4.1 summarizes LCCA input variables and the general basis used to determine their values.

Table 4.1. LCCA input variables.

LCCA Component	Input Variable	Source
Initial and Future Agency Costs	Preliminary Engineering	Estimate
	Construction Management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of Costs	Pavement Performance	Projection
User Costs	Current Traffic	Estimate
	Future Traffic	Projection
	Hourly Demand	Estimate
	Vehicle Distributions	Estimate
	Dollar Value of Delay Time	Assumption
	Work Zone Configuration	Assumption
	Work Zone Hours of Operation	Assumption
	Work Zone Duration	Assumption
	Work Zone Activity Years	Projection
	Crash Rates	Estimate
	Crash Cost Rates	Assumption
NPV	Discount Rate	Assumption

Traditionally, this uncertainty is often ignored in an LCCA. For example, the analyst may make a series of *best guesses* of the values for each input variable and compute a single *deterministic* result. The problem with this approach is that it often excludes information that could improve the decision.

In some cases a limited sensitivity analysis may be conducted whereby various combinations of inputs are selected to *qualify* their effect on analysis results. However, even with a sensitivity analysis, this deterministic approach to LCCA often conceals areas of uncertainty that may be crucial to decision-making process. Often, stakeholders seize upon the uncertainty associated with LCCA inputs and vigorously debate the validity of the results. Traditional, deterministic-based LCCA results such as these generate endless debate over which alternative truly has the lowest life-cycle cost. This process encourages division and unproductive debate.(22)

The need to make strategic long-term investment decisions under short-term budget constraints is forcing SHAs to consider risk as a criterion for judging a course of action. Risk analysis exposes areas of uncertainty for the decision maker. Based on this new information, the decision maker has the opportunity to take mitigating action to decrease exposure to risk. With the emergence of user-friendly computer software, an SHA can easily integrate quantitative risk analysis concepts into the decision-making process

GENERAL APPROACH

Figure 4.1 illustrates the basic risk analysis approach in LCCA. It shows the NPV formula typically used as the economic indicator in an LCCA. As shown in figure 4.1, a risk analysis approach uses random sampling from probabilistic descriptions of uncertain input variables

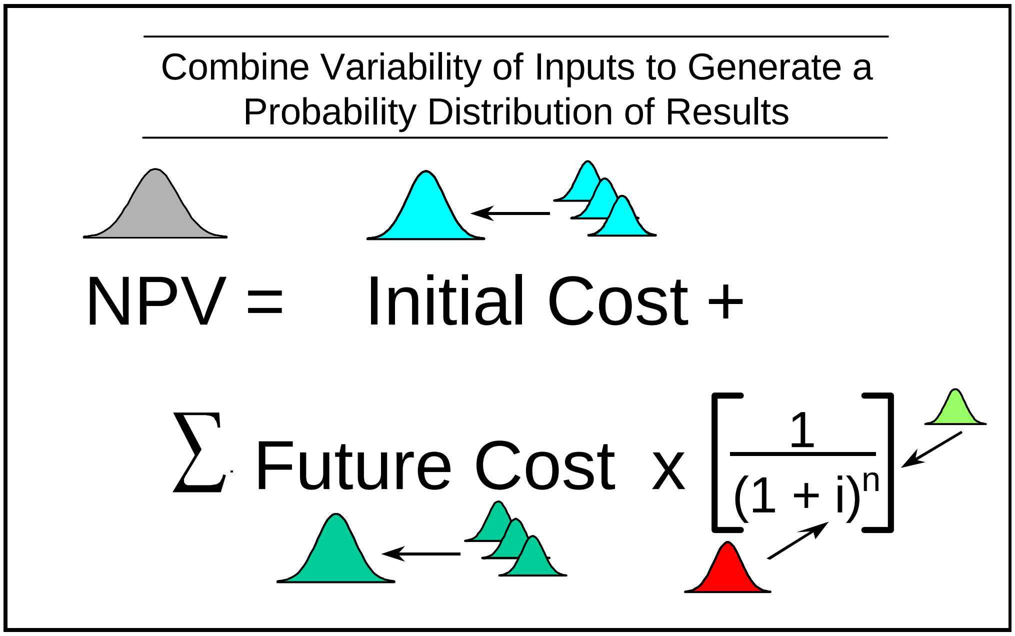

Combine Variability of Inputs to Generate a
Probability Distribution of Results
NPV = Initial Cost +
∑ Future Cost x [1 / (1 + i)^n]

Figure 4.1. Computation of NPV using probability and simulation.

(initial cost, future cost, discount rate, and year of rehabilitation) to generate a probabilistic description of results. By performing the Monte Carlo computer simulation, thousands, even tens of thousands of samples are randomly drawn from each input distribution to calculate a separate *what-if* scenario. With the speed and memory of today's personal computers, such iterations can be conducted in a matter of minutes or even seconds. The results generated from each what-if iteration are captured for later statistical analysis.

Similar to the inputs, risk analysis results are presented in the form of a probability distribution that describes the range of possible outcomes along with a probability weighting of occurrence. In other words, by randomly drawing samples from the model's input distributions, the computer combines the variability inherent in the inputs and summarizes it in the form of a probability distribution. With this information, the decision maker knows not only the full range of possible values, but also the relative probability of any particular outcome actually occurring. This is exactly the information that the decision maker needs in order to make an educated decision.

By including all possible values for the analysis inputs in relation to their probability of occurrence, risk analysis elevates the LCCA debate from contesting the *validity of results* to deciding best public policy. Armed with this new information, risk analysis provides the decision maker with the opportunity to take mitigating action to decrease exposure to risk. Moreover, risk analysis provides those vested with the appropriate authority and accountability, namely executives and elected officials, the opportunity to make decisions about taking risk.(23)

Although the risk analysis approach is calculation intensive and would be impractical to complete by hand, the approach can be easily incorporated into deterministic analysis by using user-friendly computer software programs. Today, software is available that includes advanced risk analysis techniques such as Monte Carlo simulation. Risk analysis software can be stand-alone programs (Microsoft Visual Basic, C++, etc.) or simple *add-ins* to spreadsheets such as Microsoft Excel or Lotus.

Because of the flexibility that spreadsheets offer in solving a wide variety of problems this *Interim Technical Bulletin* explores the use of risk analysis in a spreadsheet environment. The two most often used spreadsheet add-in risk analysis software programs are @RISK and Crystal Ball. They are very similar in operation and capability. For presentation purpose, this *Technical Bulletin* discussion uses @RISK in conjunction with Microsoft Excel spreadsheets.

Risk analysis has been used for a number of years in other industries. As a result, a number of approaches have evolved to conduct such analysis. The following sections outlines a general approach that may be used to conduct a risk analysis. The five proposed steps are:

1. Identify structure and logic of problem.
2. Quantify uncertainty using probability.
3. Perform simulation.
4. Analyze and interpret results.
5. Make a consensus decision.

The following sections address each of these steps in detail. The example problem presented below illustrates how risk analysis may be applied in a pavement design LCCA setting. To keep the example simple, user costs are omitted.

> *A State highway agency is considering two competing pavement design alternatives, A and B. The analysis period is 35 years, routine reactive maintenance costs differences between alternatives are insignificant, and the discount rate may range from 3 to 5 percent. Average and Standard Deviations for Agency Costs, shown in table 4.2, have been developed from an analysis of recent bid records. However, because of a lack of documented field performance data, the State convened a special panel of pavement experts to develop estimates of pavement service life for each the proposed design alternatives. Table 4.3 shows these estimates. Use a probabilistic approach to evaluate the two alternatives.*

Table 4.2. Average and standard deviations for agency costs.

Cost Item	Alternative A		Alternative B	
	Average	Std Dev	Average	Std Dev
Initial Agency Cost ($ Millions)	26.5	0.75	20.0	2.5
Future Rehabilitation Cost ($ Millions)	7.0	0.5	6.0	1.0

Table 4.3. Estimates of pavement service life.

	Alternative A			Alternative B		
	Min	Most Likely	Max	Min	Most Likely	Max
Initial Pavement Design (Years)	20	25	30	12	15	18
Future Rehabilitation Design (Years)	10	13	15	5	7	10

Step 1. Identify the Structure and Layout of the Problem

The first step in conducting a risk analysis is to identify the structure and layout of the problem. This usually involves reducing the problem to its most basic elements and describing it in the form of an analytical model. Flow charts sometimes supplement the model and clarify relationships between variables. In this example problem, the model is defined by the formula for NPV:

$$NPV = InitialCost + \sum FutureCosts\left(\frac{1}{(1+i)^{n}}\right)$$

This model may be programmed in a spreadsheet or a stand-alone program may be developed using Visual Basic, C++, etc.

Figure 4.2 shows the pavement service life curves for each alternative strategy based on the *mean* service life values from table 4.3. Based on the mean pavement service life, Alternative A

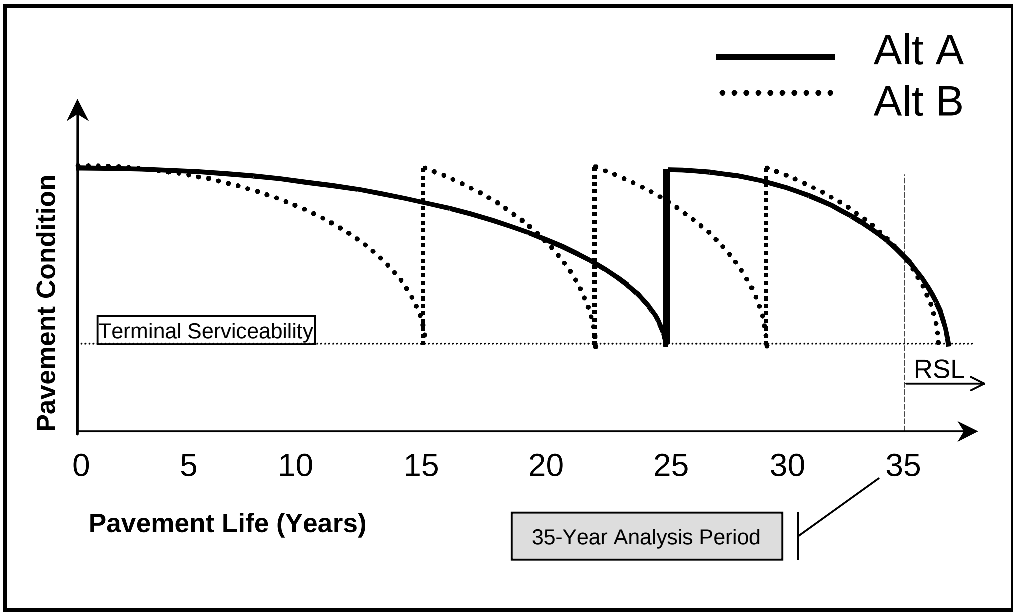


Figure 4.2. Pavement life curves for Alternatives A and B.

requires one rehabilitation; whereas, Alternative B requires three rehabilitations over the analysis period of 35 years. Figure 4.3 shows the expenditure streams for each alternative based on the *mean* service life values from table 4.3.

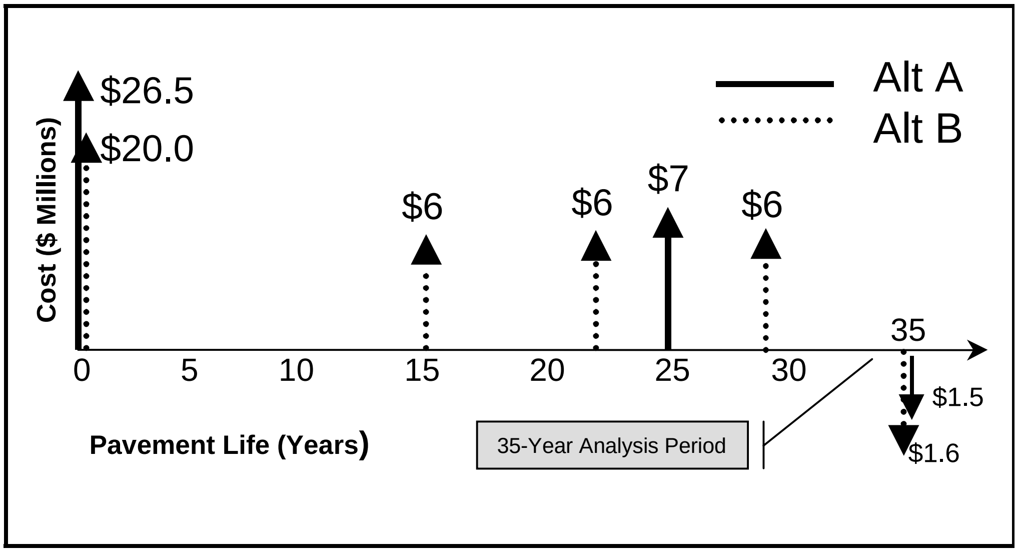


Figure 4.3. Cash flow diagram for Alternatives A and B.

Based on the mean values of estimated service lives, both Alternatives A and B have remaining service life (RSL) at the end of the 35-year analysis period. The value of the RSL must be properly accounted for and appropriately discounted. In a deterministic LCCA, treatment of the RSL of each alternative would be relatively straightforward. In the probabilistic risk analysis approach, the RSL issue becomes more complex because of the range of possible input values and the random sampling of model inputs.

The NPV model must take into account the entire range of probable pavement service lives for both the initial design and future rehabilitation designs. As such, there is a discrete probability that the last rehabilitations (as defined using mean service life values) shown in figure 4.2 could take place earlier than the mean service life values would indicate. At the same time there is a discrete probability that the last rehabilitations shown might well only be required at some point beyond the end of the analysis period. As a result, probabilistic-based risk analysis models must account for all the possibilities in determining the number, timing, and RSL of future rehabilitation requirements. Typically, logic *IF* statements provided in most spreadsheet programs or programming languages can be used to facilitate discounting future costs over the entire range of probable service life values.

In developing the structure and layout of the model, it is crucial to identify dependencies among uncertain variables to avoid producing incorrect results. Because user costs have been omitted in the example problem, the number of variables has been significantly reduced. Initial cost, future costs, discount rate, and year of rehabilitation activity are the uncertain variables in this model. The next step is to describe their uncertainty.

Step 2. Quantify Uncertainty Using Probability

The next step is to develop probability distributions for the uncertain variables identified in the previous step. A probability distribution allows the analyst to describe the complete range of values the variable may assume and weights their likelihood of occurrence accordingly.

Types of Probability Distribution

Figure 4.4 illustrates some of the more common probability distributions. Those shown include triangle, normal, and uniform distributions in histogram format.

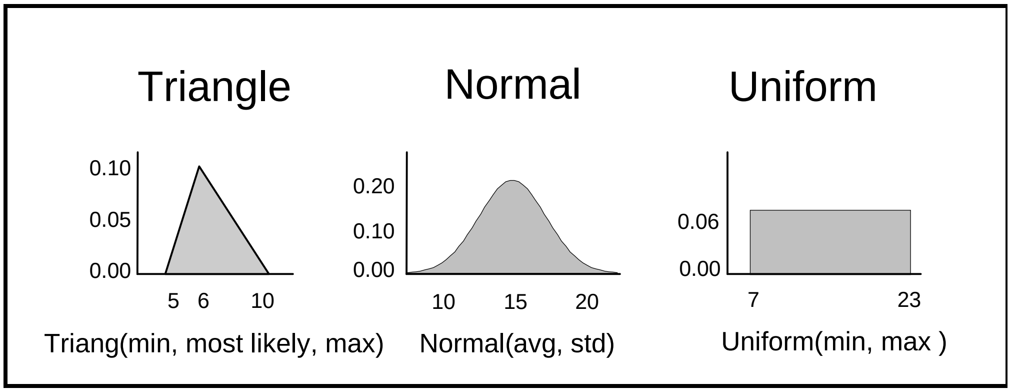


Figure 4. 4. Example probability distributions.

The horizontal axis provides a range of possible values and the vertical axis provides a relative frequency weighting of the occurrence of any particular value. For the histograms shown, the probability is equal to the area under the curve and the total shaded area is equal to 1.0.

Figure 4.5 shows a cumulative ascending probability distribution. In this case the cumulative probability is read directly off the vertical axis. For the example shown, there is a 75 percent probability that project costs will be less than or equal to $9 million. Sometimes, the cumulative distribution is shown in the descending format, in which case the probability represents the probability of exceeding any particular value.

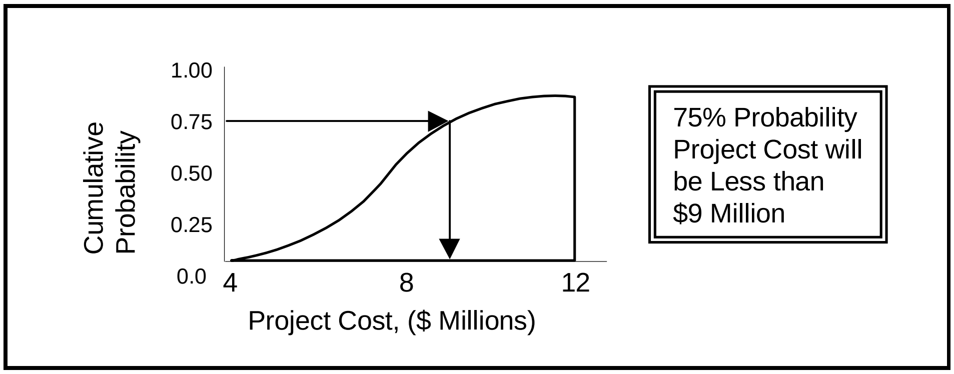


Figure 4. 5. Ascending cumulative probability distribution.

Discrete and Continuous Variables

Probability distributions may be used to model discrete or continuous variables. Discrete variables are countable. Continuous variables are not. For example, time passes gradually; whereas the numbers on a digital wristwatch change abruptly. In this case, time is a continuous variable, whereas, the watch's measurement of time is not. In practice, a continuous distribution may be used to model a discrete variable, as long as the difference between allowable values is small. For example, project costs is a discrete variable with steps of 1 cent. However, it may be modeled using a continuous distribution because the magnitude of agency cost are relatively large in a typical pavement design LCCA.

Developing Probability Distributions

Probability distributions may be developed using either *objective* or *subjective* methods. The objective method uses hard data (such as bid price list and observed capacity) to formulate the distribution; the subjective method uses expert opinion.

When existing data is available, statistical analysis packages can be used to automatically fit the probability distribution to the data. These programs compare the more common distribution types with available data. Along with recommendations on the distribution types that best describe the variability of the data, many distribution fitting programs provide statistical indicators such as Chi-squared, Anderson-Darling (A-D), and Kolmogorov-Smirnoff (K-S) that describe the goodness of fit. These statistics indicate how closely the probability distribution fits the data.

Figure 4.6 illustrates the use of group interviews for developing subjective probability descriptions of uncertain variables. As shown, expert panels are convened to establish the boundaries and general shape of input distributions. This process of eliciting information from experts is similar to the well-known Delphi method. Such meetings are structured to elicit all expert opinion. Background information is provided to participants by using scatter plots, trend charts, basic statistics, and histograms to summarize available data. Discussion topics include not only the over all uncertainty of the input variables, but also the possible interrelationships and

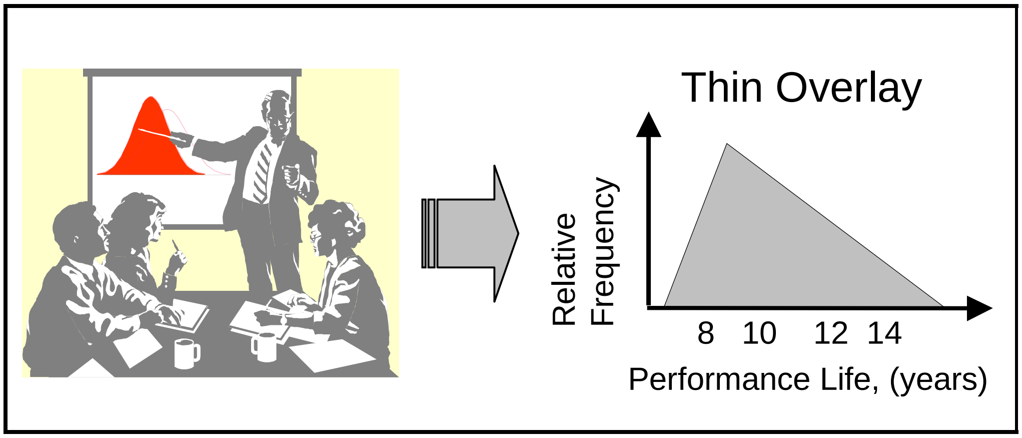


Figure 4. 6. Using expert opinion to develop probability distributions.

codependencies among the input variables. Follow-up activities may include individual interviews with meeting participants to ensure all opinions are included. Formal surveys and questionnaires may also be used in the process to ensure that the resulting distribution covers the entire range of possible values. To facilitate buy-in on model results, it is important to emphasize involving *all* stakeholders in this process.

The six distributions most often used to model the opinion of experts include:

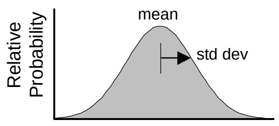
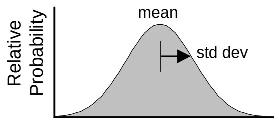
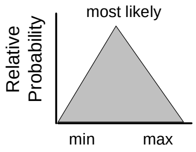
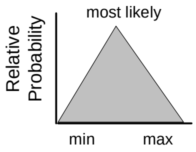
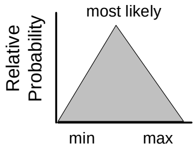
- **Triang** — No tail values.
- **Trigen** — Allow for tail values.
- **Normal** — If the analyst believes the data to be normally distributed, a technique is to back into the distribution, given the mean, min, and max values. The Empirical rule indicates that plus or minus 2 standard deviations approximate 95 percent of the data. The equation to estimate the standard deviation is: (Max – Min)/4.
- **General** — This distribution is very flexible because it allows the expert task group to tailor the shape of the curve.
- **Uniform** — A gross estimating tool. A problem with the uniform distribution is that outside the min and max values, the probability precipitously drops to 0.
- **Discrete** — To model known probabilities or to weight expert opinions.

Selecting Distributions and Defining Parameters

Returning to the example problem, the normal distribution will be used to model the variability for agency costs using the average and standard deviations provided in table 4.2. When using hard data to determine the type of distribution to use, it is important to check the reasonableness of the *goodness of fit* statistics associated with the selected distribution. When *measured* data is *not* available, a triangular distribution may be used as a rough estimate of the distribution's shape. For the example problem, a triangular distribution will be assumed and the data on minimum, most likely, and maximum service life data in table 4.3 will be used for the distribution input parameters.

If nothing were known about the shape of a distribution, it might be more appropriate to use the uniform distribution and thereby give equal weight to probability of all input values. Little is known about the variability of the discount rate in the example problem other than it ranges from 3 to 5 percent. However, recalling the OMB discount rates shown in table 1.1, a symmetrical triangular distribution with a minimum, most likely, and maximum of 3, 4, and 5 percent will be used to estimate the actual distribution. The symmetry of the triangular distribution has the added advantage of not giving advantage to a particular alternative based on the influence of higher or lower discount rates. Typically, lower discount rates favor higher initial costs and lower future costs. Higher discount rates typically favor lower initial costs and higher future costs. Table 4.4 summarizes the input distributions that will be used in the analysis.

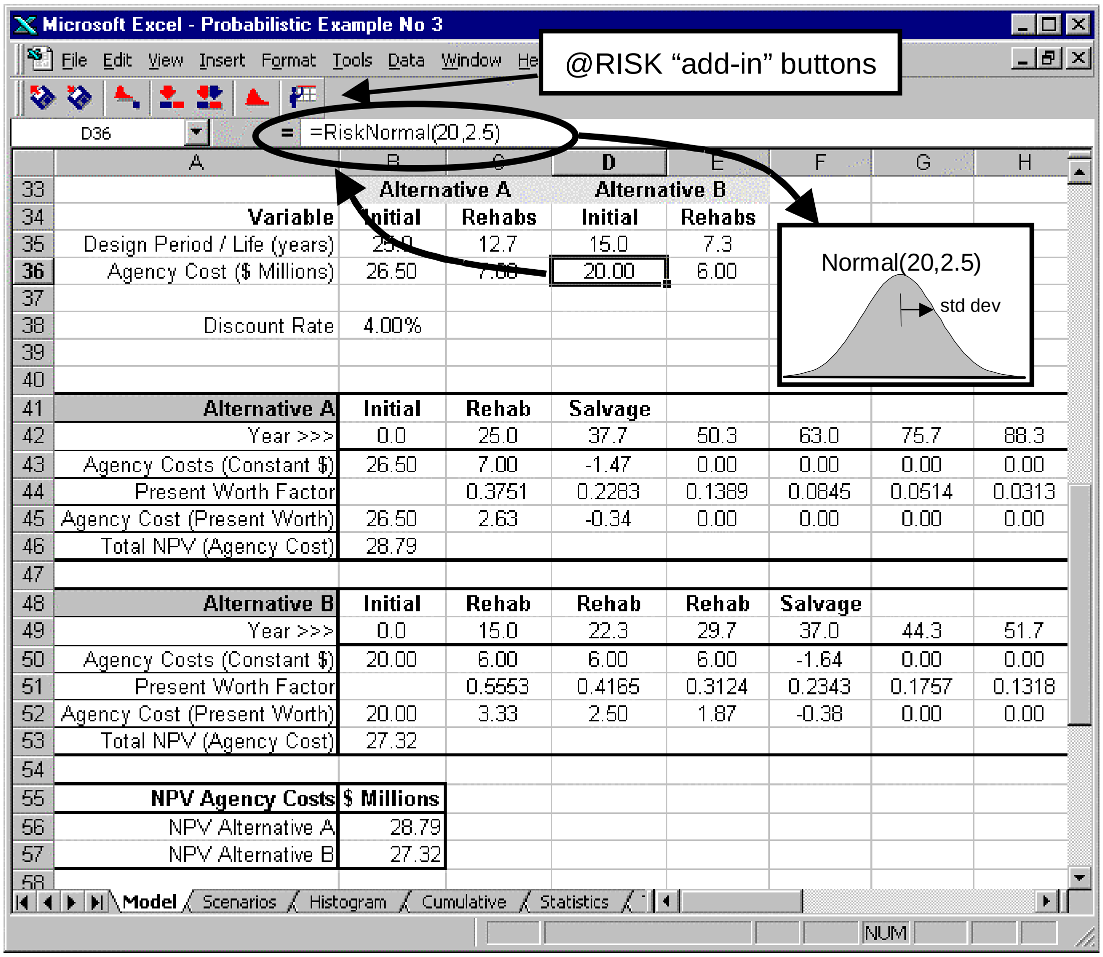
Table 4. 4. Summary of input distributions for LCCA.

Variable	Distribution Type	Distribution Type and Controlling Parameters	Illustration
Initial Agency Cost	Normal	Normal(mean, std dev) Alt A – Normal(26.5, 0.75) Alt B – Normal(20,2.5)	 Relative Probability mean std dev
Future Rehab Cost	Normal	Normal(mean, std dev) Alt A – Normal(7, 0.5) Alt B – Normal(6,1)	 Relative Probability mean std dev
Pavement Service Life – Initial Constr.	Triangular	Triang(min, most likely, max) Alt A – Triang(20,25,30) Alt B – Triang(12,15,18)	 Relative Probability most likely min max
Pavement Service Life – Rehabilitation	Triangular	Triang(min, most likely, max) Alt A – Triang(10,13,15) Alt B – Triang(5,7,10)	 Relative Probability most likely min max
Discount Rate	Triangular	Triang(min, most likely, max) Alt A – Triang(3,4,5) Alt B – Triang(3,4,5)	 Relative Probability most likely min max

Incorporating Probability Distributions in Spreadsheet Models

Probability distributions can be directly incorporated into spreadsheet models using custom distribution functions available as part of the @RISK and Crystal Ball software programs. Each of the programs includes approximately 30 additional spreadsheet functions that are add-ins to the existing spreadsheet's standard function set. Each of these additional functions represents a probability distribution (NORMAL, BETA, TRIANG, etc.).

Figure 4. 7 shows a Microsoft Excel/@RISK spreadsheet model of the example problem. It illustrates how the initial agency cost for Alternative B, shown in cell D36, can be programmed as an @RISK distribution function. Similar to a standard Excel function, an @RISK function is entered into the cell by including both the function name and argument list. Figure 4.7 shows initial agency cost represented as a normal distribution with a mean of $20 million and a standard deviation of $2.5 million.



Microsoft Excel - Probabilistic Example No 3

D36 = =RiskNormal(20,2.5)

	A	B	C	D	E	F	G	H
33		Alternative A		Alternative B				
34	Variable	Initial	Rehabs	Initial	Rehabs			
35	Design Period / Life (years)	25.0	12.7	15.0	7.3			
36	Agency Cost ($ Millions)	26.50	7.00	20.00	6.00			
37								
38	Discount Rate	4.00%						
39								
40								
41	**Alternative A**	**Initial**	**Rehab**	**Salvage**				
42	Year >>>	0.0	25.0	37.7	50.3	63.0	75.7	88.3
43	Agency Costs (Constant $)	26.50	7.00	-1.47	0.00	0.00	0.00	0.00
44	Present Worth Factor		0.3751	0.2283	0.1389	0.0845	0.0514	0.0313
45	Agency Cost (Present Worth)	26.50	2.63	-0.34	0.00	0.00	0.00	0.00
46	Total NPV (Agency Cost)	28.79						
47								
48	**Alternative B**	**Initial**	**Rehab**	**Rehab**	**Rehab**	**Salvage**		
49	Year >>>	0.0	15.0	22.3	29.7	37.0	44.3	51.7
50	Agency Costs (Constant $)	20.00	6.00	6.00	6.00	-1.64	0.00	0.00
51	Present Worth Factor		0.5553	0.4165	0.3124	0.2343	0.1757	0.1318
52	Agency Cost (Present Worth)	20.00	3.33	2.50	1.87	-0.38	0.00	0.00
53	Total NPV (Agency Cost)	27.32						
54								
55	**NPV Agency Costs**	**$ Millions**						
56	NPV Alternative A	28.79						
57	NPV Alternative B	27.32						
58								

Figure 4. 7. Excel spreadsheet showing @RISK add-in buttons.

Distribution functions such as this may be used anywhere in the spreadsheet where there is uncertainty about the value of the variable. @RISK functions may be used the same way that any of the other normal spreadsheet functions are used. They may be included in mathematical expressions having cell references or be used as part of IF–THEN expressions, clarifying logical relationships.

Step 3. Perform Simulation

After the structure of the model has been established and distributions developed and substituted for the uncertain input variables, the next step is to run a simulation of the model to obtain results. A simulation is essentially a rigorous extension of a sensitivity analysis that uses different randomly selected sets of values from the input probability distributions to calculate separate discrete results. The results are arrayed in the form of a distribution covering all possible outcomes. This process of using random numbers to sample from probability distributions is known as Monte Carlo sampling.(24) Figure 4.8 depicts the process of sampling from an input distribution.

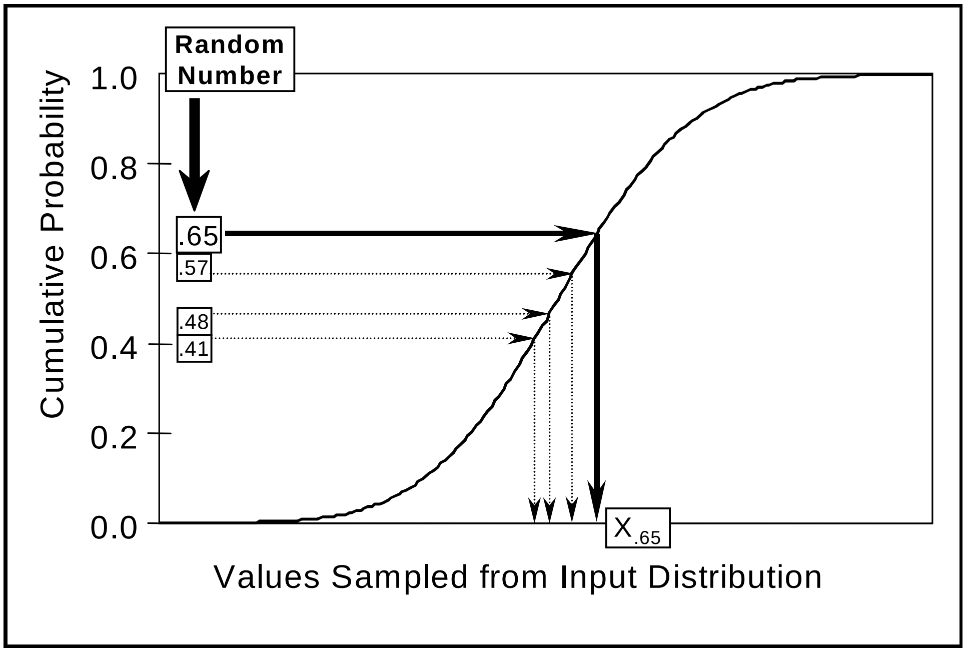


Figure 4.8. Monte Carlo sampling showing four iterations.

As shown in figure 4.8, a series of random numbers between 0 and 1 are generated by the computer along the cumulative probability scale of the input distribution. Values corresponding to each random number are sampled along the x-scale. For the example shown, when the computer generates the random number 0.65 in figure 4.8, a corresponding value of $x_{.65}$ is sampled. The sampled value is then combined with other distribution samples to compute a single result. It is important to note that the computer uses a *uniform* distribution to generate the random numbers and all values along the cumulative scale of the y-axis have equal probability of being selected. Therefore, x-axis values corresponding to portions of the distribution curve where the slope is steeper (more vertical) have a greater likelihood of being sampled compared to x-axis values that correspond to portions of the curve with flatter slopes.

In a simulation, each iteration represents a possible scenario or outcome. The results of each iteration are captured, compiled, and subjected to rigorous statistical analysis. This process of sampling from a probability distribution is repeated until the specified number of iterations are completed or until the simulation process converges (i.e., the point at which additional iterations do not significantly change the output distribution).

Simulation techniques, such as the Monte Carlo, typically require a large number of iterations to ensure sufficient opportunity to sample low probability values. This is especially true when highly skewed distributions are used to describe the input variables. When the number of iterations performed is low, *clustering* can occur. In this situation, sampled values are tightly clustered around high probability outcomes and the low probability outcomes may not be adequately sampled. Clustering is particularly important when low probability outcomes can have an inordinately serious effect on results. Such outcomes must be accounted for in the simulation, but, to do this, they must be sampled. This has led to the development of a sampling technique known as Latin Hypercube, which forces a more representative sampling with a much lower number of iterations.(25,26,27)

Latin Hypercube is a stratified sampling technique where the probability scale of the cumulative distribution curve is divided into an equal number of probability ranges. The number of ranges used is equal to the number of iterations performed in the simulation.

Figure 4.9 illustrates Latin Hypercube sampling, in this case, showing four iterations, and therefore four distinct divisions of the cumulative probability scale. As shown in figure 4.9, within a division (in this case, division number 2), a random number between 0 and 1 is generated and serves as the basis for selecting the input value. In the case shown, 0.4 is generated and a

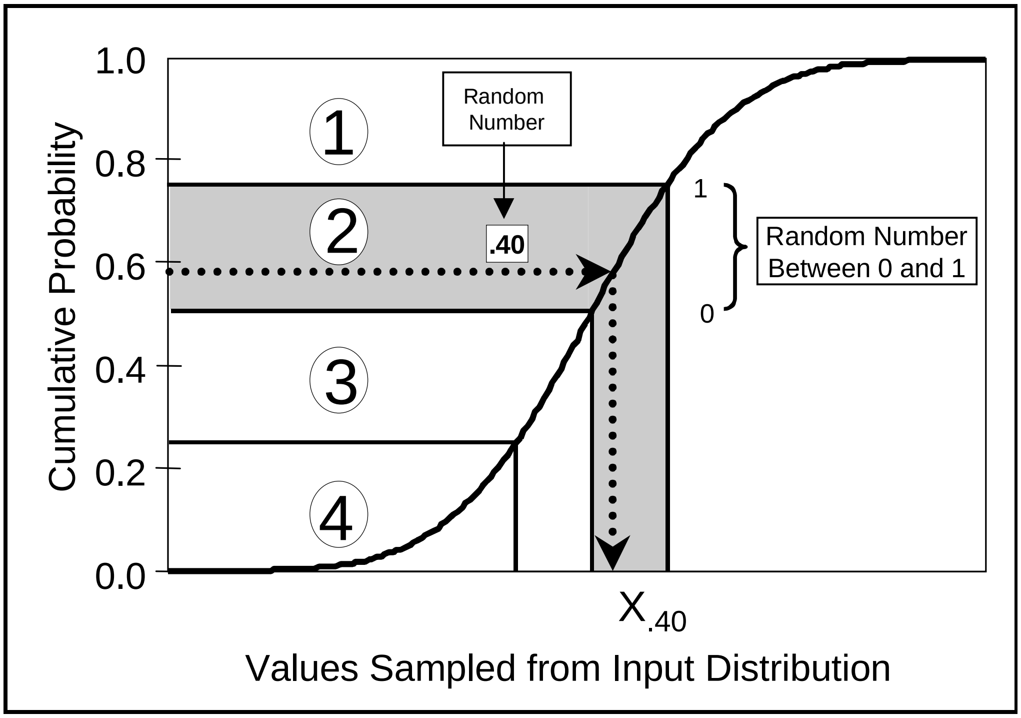


Figure 4.9. Latin Hypercube sampling showing four iterations.

corresponding value is sampled from the distribution. In future iterations, the previously sampled section is not sampled again. Because the Latin Hypercube sampling technique *memorizes* the sections it sampled, it therefore achieves convergence much more quickly when compared to the Monte Carlo. In addition to being more efficient, this technique ensures low probability values are sampled and included in the simulation.

Figures 4.10 and 4.11 compare the effect of Monte Carlo and Latin Hypercube sampling when the number of iterations is limited to 100. As shown, 100 samples were randomly drawn from the normal distribution function for Initial Agency Cost Alternative B — Normal(20, 2.5). Figure 4.11 shows that the stratified Latin Hypercube sampling technique achieves a distribution

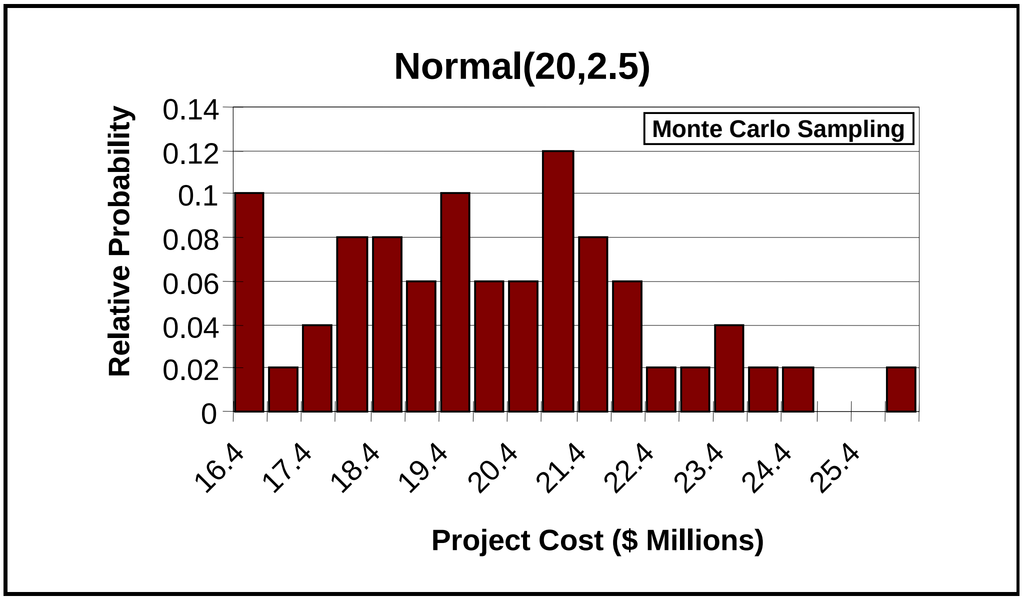


Figure 4.10. Monte Carlo sampling – 100 iterations.

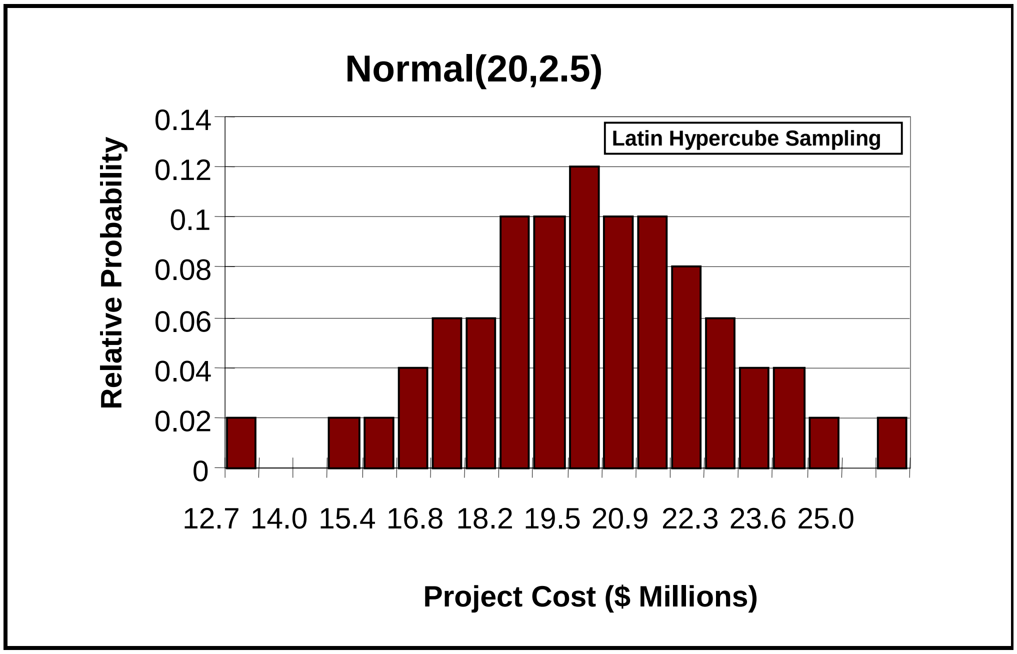


Figure 4.11. Latin Hypercube sampling – 100 iterations.

that more resembles the familiar bell-shaped normal curve when compared to pure random Monte Carlo sampling. Because of stratified sampling, it is possible to achieve convergence in fewer iterations as compared to the Monte Carlo simulation.

A cardinal rule of risk analysis modeling is *every iteration of a risk analysis simulation must be a scenario that can actually occur.* Failure to recognize the dependent relationship between input variables is usually a prime source of error in simulation modeling. Suppose, for example, that as part of a risk model, probability distributions describe both the random variability for AADT and pavement service life. Typically, pavement engineers have related shorter pavement life to higher traffic volumes. Therefore, it would be irrational for a simulation model to sample the high side of the traffic probability distribution curve, and at the same time, sample the high side for pavement life. Formal treatment of this relationship between AADT and pavement service life is usually accounted for in the simulation by using a correlation matrix. Given that user costs are omitted from the example problem, the number of variables has been greatly reduced and the analyst does not have to correlate variables. Another source of error is sampling input distributions outside reasonable bounds. For example, sampling from a normal distribution for project cost may produce misleading results, particularly when the values sampled are negative. The use of logical statements as well as establishing bounds for the distribution may be used to overcome errors such as this.

Figure 4.7, previously shown, is a spreadsheet that shows the results of 10,000 iterations processed using a Microsoft Excel @RISK model.(28) The simulation run time was 47 seconds using a Pentium 166 MHz Computer with 80 MB RAM. Before analyzing the simulation results, it is important to test the assumptions to make certain the model is robust across the entire range of possible values for the uncertain inputs. The model should be tested as each component is added to make certain the results are reasonable. This is especially true for large and complex models. A visual tool to examine a model during a simulation is to construct a plot of the performance curves and cash flows. @RISK will allow a simulation to be executed one step at a time. By stepping through the simulation and viewing the changing shape of the output graphs in real time, it is possible to detect errors that may exist in the model.

Another tool that may be used to test a risk model is to incorporate the use of a random number generator seed. A *seed value* is a value that is used to start the sampling process and all subsequent random numbers will rely on this value. Providing that the model is not changed, a second run of the simulation using the same seed value will produce exactly the same results. This can be very useful in testing the effect of varying distributions on risk results. By using a seed value, the analyst can be certain that any change in the result is caused by changes in the model and is not a result of randomness of the sampling. After the model is tested and a simulation is performed, the next step is to analyze and interpret the results.

Step 4. Analyze and Interpret Results

If the analysis had been conducted according to the traditional deterministic approach, all that would be available to base the decision on would be the means of the output distributions: Alternative A $28.79 million and Alternative B $27.32 million, respectively. Based on the NPV means, it is readily apparent that Alternative B is less than Alternative A by $1.47 million (~ 4.8 percent). Some SHAs would probably consider this difference in means insignificant and may exclude lowest NPV as a consideration for selecting the alternative. Without an analysis of the variability about the mean, a decision such as this may prove to be a poor choice — depending on the decision maker's tolerance for risk.

Interpretation of risk analysis results goes beyond a simple comparison of which alternative on average costs less by including an analysis of the likelihood that any particular outcome will occur. There is no presumption that any particular alternative is better. Figure 4.12 shows the risk profile of the NPV for Alternatives A and B in histogram form, where the probability is the area under the curve. As Figure 4.12 shows, the entire range of conceivable outcomes is arrayed with the estimated probability of each outcome actually occurring. The main advantage of the histogram is that it readily shows the variability about the mean. The wider the distribution, the greater the variability. As shown, the outcome for Alternative B is more uncertain than Alternative A.

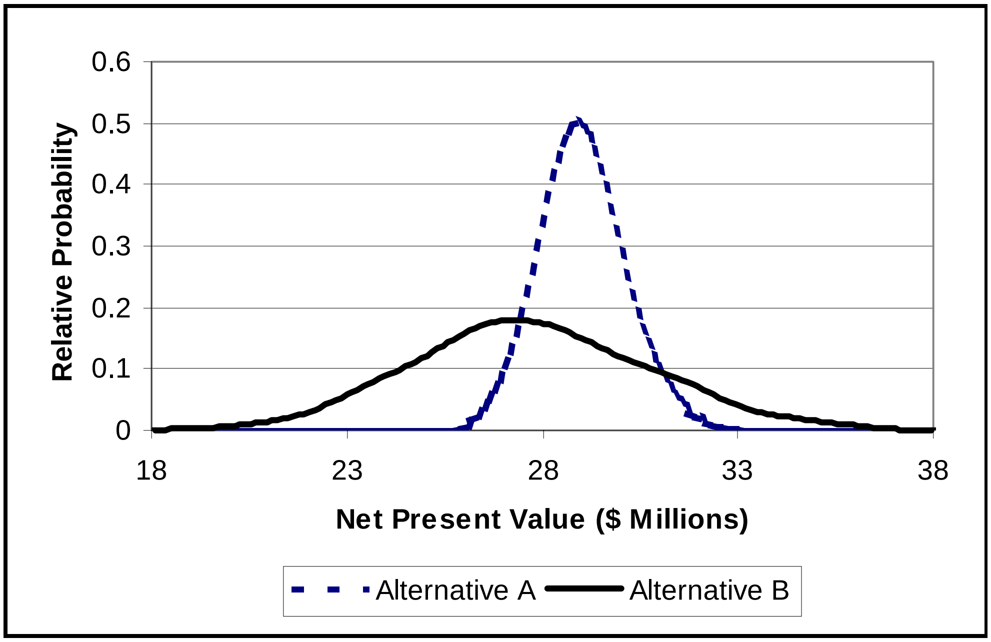


Figure 4.12. Histogram NPV for Alternatives A and B.

In interpreting the risk profile in figure 4.12, it is important to distinguish between upside risk and downside risk. Downside risk for project cost implies cost overrun — chance of financial failure. Upside risk for project costs implies cost underrun — opportunity for low cost. As figure 4.12 shows, Alternative B has greater upside risk compared to Alternative A. However, as discussed below, it is important to quantify the probability of cost overrun for Alternative B.

Figure 4.13 shows the risk profiles for Alternatives A and B in cumulative form. As shown, there is a 60 percent probability that project costs for Alternative B will be less than $28.27 million. This means that for the 10,000 iterations that were processed, 60 percent of the calculated values for NPV were less than $28.27 million. The variability for the proposed alternative is inversely proportional to the slope of the cumulative curve. In other words, the steeper the slope, the less variability. The flatter the slope, the greater the variability. As shown, the slope for Alternative B is flatter than that for Alternative A, and is therefore more variable.

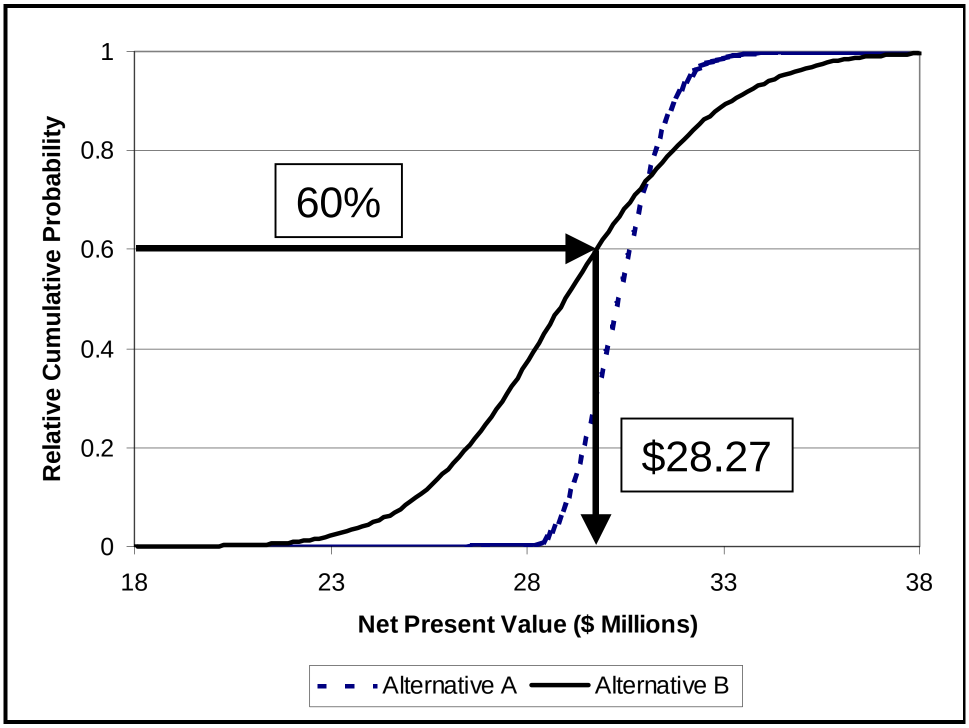


Figure 4.13. Cumulative risk profile of NPV for Alternatives A and B.

A risk analysis provides much more information than a simple deterministic solution. As table 4.5 shows, additional information comes in the form of basic statistical measures of simulation results that reveals the underlying uncertainty associated with each alternative. In interpreting the risk involved with each alternative, it is important to identify the magnitude of the extremes of the distributions shown in figure 4.12. As table 4.5 shows, Alternatives A and B have minimums of $25.4 and $13.3 million and maximums of $33.04 and $40.35 million, respectively. The standard deviation for Alternative B is much greater than Alternative A. Analysis of the distribution tails reveals a 10 percent probability that the NPV of Alternative B will be less than

Table 4.5. Risk profile statistics for Alternatives A and B.

	Net Present Value ($ Millions)		
Basic Statistic	**Alternative A**	**Alternative B**	**B – A**
Minimum	$ 25.40	$ 13.13	$ (12.27)
Maximum	33.04	40.35	7.31
Mean	28.93	27.60	(1.33)
Std Deviation	1.04	3.13	2.09
Percentile			
5%	27.33	22.66	(4.67)
10%	27.65	23.71	(3.94)
15%	27.88	24.39	(3.49)
20%	28.05	24.98	(3.07)
25%	28.22	25.50	(2.72)
30%	28.36	25.93	(2.43)
35%	28.49	26.34	(2.15)
40%	28.62	26.73	(1.89)
45%	28.75	27.12	(1.62)
50%	28.87	27.48	(1.40)
55%	29.00	27.87	(1.13)
60%	29.13	28.27	(0.86)
65%	29.27	28.71	(0.56)
70%	29.42	29.13	(0.29)
75%	29.58	29.63	0.06
80%	29.76	30.20	0.44
85%	30.01	30.88	0.87
90%	30.31	31.67	1.35
95%	30.75	32.98	2.23

Alternative A by as much as $3.94 million. Conversely, there is a 10 percent probability that Alternative B will exceed the cost of Alternative A by $1.35 million. However, 70 percent of the time, the NPV for Alternative B was less than Alternative A. It is interesting to point out that the mean shown in table 4.5 for Alternatives A and B is slightly different from the expected values shown in figure 4.7. This is because table 4.5 means are based on a simulation of 10,000 iterations.

As part of the risk assessment, a sensitivity analysis can be performed on simulation results to identify significant input variables that are important in determining the output distributions. The results of this analysis are usually displayed in the form of a Tornado plot, as shown in figures 4.14 and 4.15. The higher the correlation coefficient, the more significant the input variable is on determining the results. The variables listed at the top of the graph are more significant than those at the bottom. The degree of correlation may be calculated using either the rank order correlation or stepwise least squares regression. Rank order correlation makes no presumption

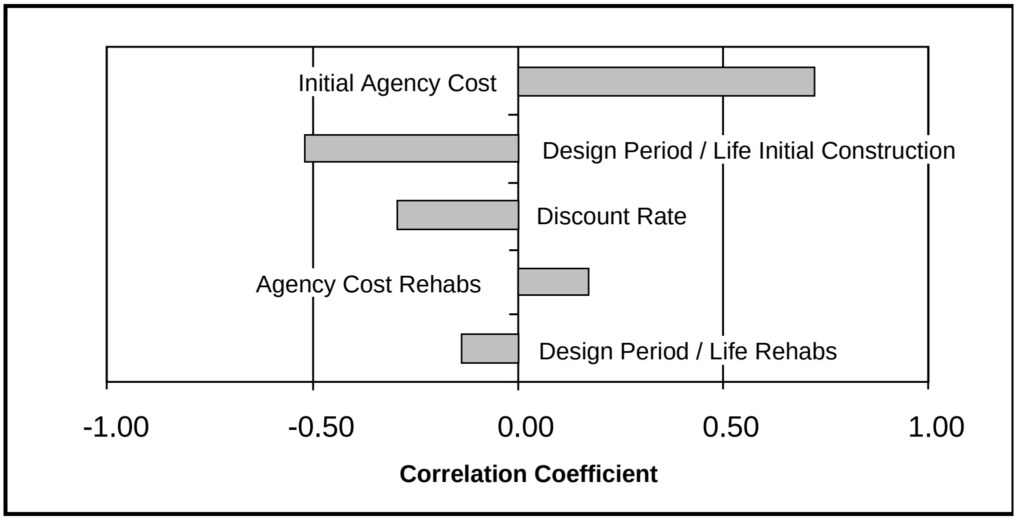


Figure 4.14. Correlation sensitivity plot for NPV Alternative A.

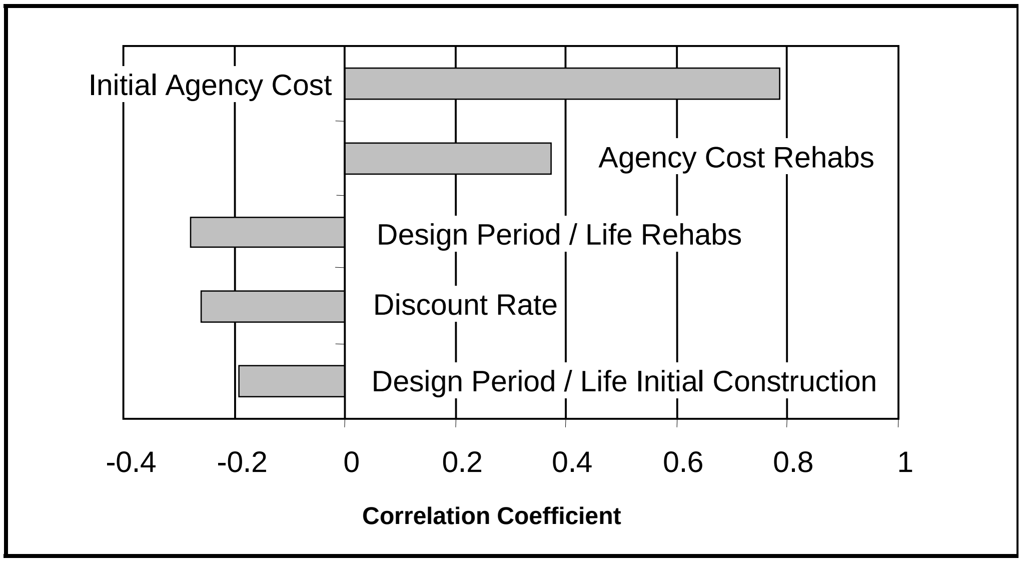


Figure 4.15. Correlation sensitivity plot for NPV Alternative B.

about the relationship between the input and output variables. Least squares regression assumes a linear relationship between input and output variables. This is important to note because models that incorporate divisions and power functions often violate this later assumption. Both figures 4.14 and 4.15 use rank order correlation. Typically, correlation coefficients less than about 0.6 are not significant.

Figure 4.14 shows *Initial Agency Cost* has a correlation coefficient of 0.72. This means that if *Initial Agency Cost* moves one standard deviation (in either direction), then the NPV for Alternative A will move 0.72 of a standard deviation in the same direction. If *Design Period/*

Life Initial Construction moves one standard deviation (in either direction), then expectations are that the NPV for Alternative A will move 0.52 standard deviations in the opposite direction, because the relationship is reversed as indicated by the negative correlation coefficient.

To further the evaluation, the analyst can explore the low probability area of the outcome distribution curve for Alternative B, where the NPV is greater than that for Alternative A, (i.e., greater than the 90th percentile). Figure 4.16 illustrates the need to identify the key input variables that produce low probability scenarios — the extremes, or tails, of the results distribution for Alternative B.

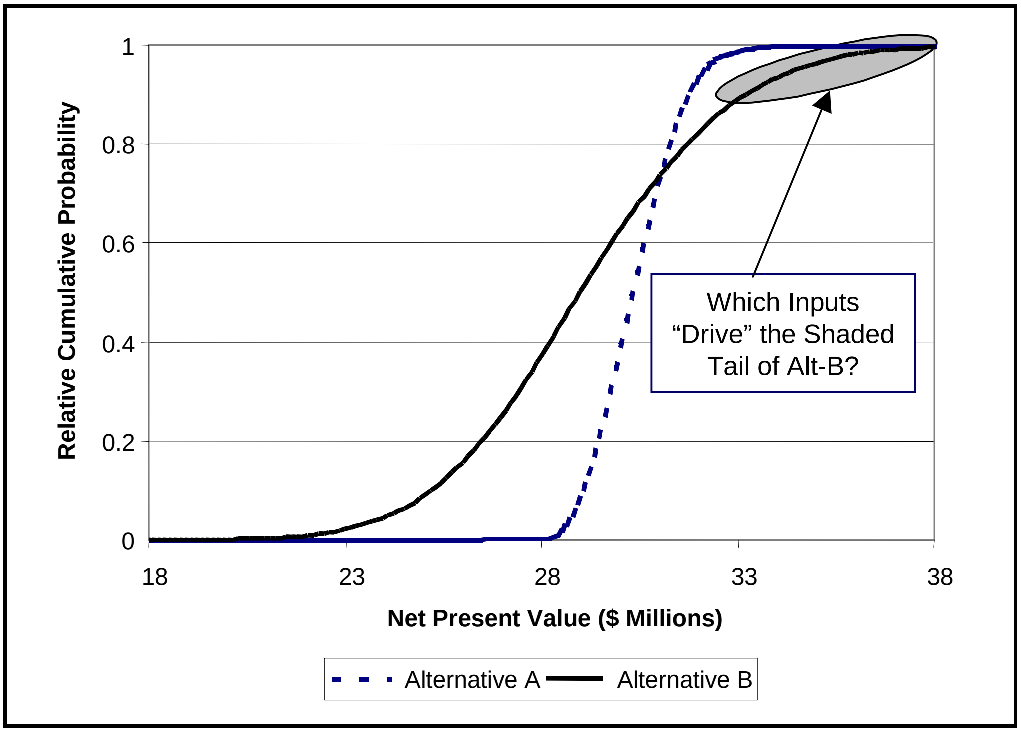


Figure 4.16. Analysis of distribution tails.

A *scenario analysis* can identify those variables that may cause a project to have significant cost overrun. The analysis is easily accomplished using Excel/@RISK, which uses the following procedure to identify *significant* inputs for a particular scenario.

1. Each input variable that affects the selected output is found.
2. The median and standard deviation of each input is calculated.
3. A *subset* is created containing only the iterations in which the output achieves the defined target.
4. The median of each input found in step 1 is calculated for the subset data.
5. For each input found in step 1, the difference between the simulation median and the subset median is calculated and compared to the standard deviation of the input data. If the absolute value of the difference in medians is greater than 0.5 of the standard deviation of the whole, then the input variable is deemed *significant* — otherwise the input is ignored.

Table 4.6 identifies the significant inputs in the upper 10 percent of the distribution for Alternative B as *Design Period/Life Initial Construction, Agency Cost Initial Construction, and Agency Cost Rehabilitation*. Now that the *drivers* in the tail are known, the decision maker may choose to take some mitigating action against these significant inputs to reduce exposure to upside risk. For example, the decision maker may choose to aggressively control agency costs.

Table 4.6. Scenario analysis results for NPV Alternative B.

Significant Inputs When NPV Alternative B Greater Than 90th Percentile	Median of Samples in Subset Iterations Meeting Target
Design Period/Life Initial Construction (Years)	14.1
Agency Cost Initial Construction ($ Millions)	21.8
Agency Cost Rehabs ($ Millions)	5.43

Step 5. Make Consensus Decision

In order to make a decision based on risk analysis results, it is important for the decision maker to define the level of risk the organization can tolerate. Decision makers who can tolerate little risk prefer a small spread in possible results, with most of the probability associated with desirable results. Conversely, if decision makers are risk-takers, then they will accept a greater amount of spread, or possible variation in the outcome distribution. Most decision makers can reach a consensus decision after weighing the probability for upside and downside risk. In our example, clearly Alternative B appears to be the better alternative since there is far greater likelihood of cost savings compared to Alternative A. Also, the probability of cost overrun, compared to Alternative A, appears to be quite low — less than 30 percent.

The reason that this step is called a *consensus* decision is because all stakeholders were involved in developing the input probability distributions early in the process. As a result, this overall process should diffuse any negative reaction special interest groups may have with the outcome. The result is that the entire risk analysis process facilitates consensus building among stakeholders so that action, in the best public interest, may be taken.

PRESENTING RISK ANALYSIS RESULTS

A risk analysis is of little value if the results cannot be understood. Thus, this section offers practical advice in presenting risk analysis results to decision makers. The first step in presenting risk analysis results is to know the audience. Here are some basic questions to ask prior to presenting the results.

- Does the audience need a risk primer?
- Does the audience buy in to the risk analysis approach?
- Does the audience buy in to the analysis?

Does the audience need a risk primer? One of the primary reasons people have difficulty interpreting risk analysis results is they often have little understanding of basic statistics which is a prerequisite to interpreting risk results. Therefore, it may be necessary to provide an introductory session to train decision makers on fundamental risk analysis concepts such as Monte Carlo sampling and probability distributions before presenting risk analysis results. The purpose of such a session is to provide just enough information so the decision maker can understand and interpret risk analysis results. There is no reason to include a formal treatment of the statistical theory underlying the concepts.

Does the audience buy in to the risk analysis approach? Although simulation techniques have been around since the early 1940s, it is only recently that these techniques have been employed in the spreadsheet world of the analyst. Risk analysis is therefore a relatively new way of treating uncertainty and involves an entirely new approach to understand and interpret the results. Risk analysis is an analytical tool to aid the decision maker to select the best alternative. By exposing previously hidden areas of uncertainty critical information is revealed thereby providing the decision maker the opportunity to take mitigating action to decrease exposure to risk. Most importantly, risk analysis provides those vested with the appropriate authority, namely executives and elected officials, the opportunity to make decisions about risk taking. As a compliment to LCCA, risk analysis elevates the debate—from the validity of LCCA results—to taking action that is in the best interest of the general public.

Does the audience buy-in to the analysis? Anyone examining risk analysis results should be cautious. First and foremost is the model correct? There is nothing worse than making high stakes decisions based on a model that is wrong. It is important that the structure and logic of the model be verified by an independent party, particularly, if risk analysis becomes an integral part of the decision making process.

Does the model follow generally accepted procedures? For example, there are a number of different procedures available to determine user costs. Are the procedures that are employed documented and do stakeholders, in general, accept the procedures? This is important to obtain buy-in on risk results. How were the input distributions determined? Were stakeholders involved in the process? Does the range for the distributions fall within a generally accepted range? Finally, the level of detail in a risk analysis report should be commensurate with the problem at hand.

The presenter of a risk analysis report should be able to answer key questions regarding the risk analysis results. In the presentation, it is recommended that the developer of the risk model be present to answer key questions about the structure and logic of the model. In presenting risk analysis results it is important not to overburden the audience with statistics. A simple *1-page* summary to include summary statistics, histogram and cumulative distributions, and perhaps a Tornado graph to show the important inputs in the analysis should be sufficient. When conveying probability graphs to upper management it is highly recommended that either cumulative ascending or cumulative descending graphs be presented. The presenter should choose a presentation method and stay with it. Structure and logic charts are also quite useful in presenting the simulation process as well as discussing judgements about the model relationships. Place model assumptions at the back of the report. These are important, but not so important as to be in the summary sheet.

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APPENDIX: RESOURCES

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Project *reports are issued at the conclusion of each NCHRP research project.*

Report 133 *Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects*, 1972.

Synthesis *publications are a series of reports of studies on current practice in the field of highway transportation. Projects are sponsored by AASHTO.*

Synthesis Report 112: *Life-Cycle Cost Analysis of Pavements*, 1985.

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Synthesis Report 142: *Methods of Cost Effectiveness Analysis for Highway Projects*, 1988.

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Research Projects

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NCHRP 1-34	*Guidelines for Subsurface Drainage;* Final Report, February 1998.
NCHRP 12-43	*Life-Cycle Cost Analysis of Bridges*, 1997.
Project 2-18 FY91	*Research Strategies for Improving Highway User Cost-Estimating Methodologies*; Hickling, Louis, Brod, April 1994.
Project 2-18 (2)FY93	*Valuation of Travel—Time Savings and Predictability in Congested Conditions for Highway User Cost Estimation Procedure,* Hickling, Louis, Brod, July 1995.
Project 2-18 (3)FY94	*Development of an Innovative Highway User-Cost Estimation Procedure*; Hickling, Louis, Brod, March 1995.
Project 2-18 (4)FY95	*Proposal—Development and Demonstration of StratBENCOST Procedure*; Hickling, Louis, Brod, (anticipated completion, November 1998).
Project 7-12	*Microcomputer Evaluation for Highway User Benefits,* October 1993.
Project 10-50	*Strategies for Rehabilitation—Rigid Pavements Subjected to High-Traffic,* Draft Interim Report (Revised), Nichols Consulting Engineers, March 1998.

AASHTO

LCCA Survey, Summer 1994.

A Manual on User Benefit Analysis of Highway and Bus Transit Improvement, 1977.

The World Bank

Anthony Boardman, Aidan Vining, and W.G. Waters II, *Cost and Benefits through Bureaucratic Lens: Example of a Highway Project*.

COMPUTER SOFTWARE

Selected Sources of Software

McTrans and PC Trans

McTrans and PC Trans are microcomputer software distribution centers for transportation-related software, including public domain software such as MicroBENCOST and QueWZ, which were developed with Federal funds. For information, contact:

McTrans
phone: 352-392-0378
fax: 352-392-3224
Web site: www.mctransce.ufl.edu
email: uftrc@ce.ufl.edu

PC Trans
phone: 785-864-5658
fax: 785-864-3199
Web site: www.kuhuh.cc.ukans.edu./~pctrans.
email: pctrans@kuhuh.cc.ukans.edu.

Other Software

There are several powerful microcomputer-based risk analysis software programs currently on the market that work well in conjunction with Lotus and Microsoft® Excel spreadsheet applications. Authors of this *Technical Bulletin* have used @Risk and Crystal Ball.

@Risk
Palisade Corporation
phone: 1-800-432-7475
Web site: www.palisade.com

Crystal Ball
Decisioneering Corporation
phone: 1-800-289-2550
Web site: www.decisioneering.com

Other software programs may be commercially available.

For information on DataPave, contact FHWA Customer Support 423-481-2967. Web site is: http://DataPave.fhwa.dot.gov, http://www.LTPPDatabase.com/main.htm, email:LTPPinfo@fhwa.dot.gov